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Continuous longitudinal bar reinforcing steel in Indiana experimental concrete pavement

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BUREAU OF PUBLIC ROADS

U. S. DEPARTMENT OF COMMERCE

E. A. STROMBERG, Editor

Transportation

Continuous Reinforcement in Concrete Pavement

A Cooperative Investigation by the Bureau of Public Roads and the Indiana State Highway Commission

Reported by HARRY D. CASHELL, Highway Research Engineer, Bureau of Public Roads, and SANFORD W. BENHAM, Research Engineer, Indiana State Highway Commission

Concrete pavements, unless constructed with closely spaced transverse joints, generally crack transversely at frequent intervals. These cracks tend to open appreciably with time unless the pavement is reinforced. For some years there has been a continued interest on the part of highway engineers regarding the practicability of concrete pavements constructed without transverse joints and reinforced longitudinally with continuous bonded steel in sufficient amount to hold all cracks closed.

In the fall of 1938 a considerable number of continuously reinforced sections, ranging from 20 to 1,310 feet in length, were constructed near Stilesville, Indiana, on U S 40 as a cooperative research project to study the effects of varying amounts of longitudinal steel in sections of various lengths.

The behavior of the sections during the first 10 years of service life conclusively shows that continuous reinforcement can be depended upon to prevent the opening of transverse cracks in concrete pavements. In the long, heavily reinforced sections many fine cracks have developed in the central region. These cracks have not opened and have raveled only slightly with traffic and exposure, a condition that has required no maintenance and may be considered superficial. The sections have remained strong, durable structural units.

The presence of even the heaviest longitudinal bar reinforcement has apparently not affected adversely the condition of the concrete in the pavement. The concrete appears to be sound throughout, there has been no spalling, and there is a complete absence of longitudinal cracking above the bars. In fact, the manner in which the steel has held closed all cracks, especially those in the more heavily reinforced sections, is believed to have been conducive to distributed interfacial pressure at the cracks which should tend to minimize damage to the concrete from concentrations of pressure such as sometimes develop at cracks in plain concrete pavements.

Pumping has developed at many of the transverse joints but, with two exceptions, has not been observed at any of the vast number of transverse cracks. This indicates that a concrete pavement without transverse joints and containing adequate longitudinal reinforcement is not nearly so susceptible to pumping as pavements of other designs.

In spite of the many transverse cracks that have developed in the long sections, the riding quality of the pavement has remained excellent and the pavement itself has been protected from damaging impact forces such as tend to develop where the surface alinement is not maintained.

SINCE 1938 the Bureau of Public Roads and the State Highway Commission of Indiana have cooperated in making detailed observations of an experimental concrete pavement containing a wide range of continuously reinforced sections. Three published reports[1] have described the scope of the study, the construction of the project, and the observed behavior of the pavement during the first 5 years of service. The present report, which may be considered the major report of the investigation, describes the performance of the various sections over a period of 10 years. Although previously published information is avoided as much as possible, certain essential data are repeated here for both clarity and completeness of the report.

The experimental pavement is a 9–7–9-inch thickened-edge type, 20 feet wide and approximately 6 miles long. It is located near Stilesville, about 30 miles west of Indianapolis, and was constructed during September and October of 1938 as part of the eastbound lanes of the divided highway U S 40.

Traffic counts indicated an average annual daily volume (in both directions) of 3,500 vehicles in 1941 when the pavement was 3 years old. Of these, 1,125 were trucks and busses, the maximum daily gross load being at least 48,000 pounds. In 1948, when the pavement had been in service for 10 years, the average annual daily traffic volume had increased to 5,100 vehicles, trucks and busses comprising 1,280 of the total. At this time the maximum daily gross and axle loads were at least 51,000 and 20,400 pounds, respectively.

Briefly, the experimental pavement consists of sections ranging in length from 20 to 1,310 feet. Incorporated in these sections are various amounts of steel for each of three types of reinforcement. The number and range in length of the individual sections, together with pertinent data on the reinforcing steel used in each, are given in table 1 (p. 4). The lengths of the sections necessary to develop the steel stresses shown in the table were calculated on the basis of certain assumptions as to the resistance offered by the subgrade as the pavement expands and contracts. It was assumed that the resistance would be constant and could be expressed as a coefficient equal to 1½ times the weight of the pavement.

The maximum steel stresses were intended to be such that the elastic limit of the reinforcement would be approached in the longest section of each group, producing, under repeated stressing, inelastic elongation with a consequent opening of the cracks.

Since a wide range in slab end movements was expected in the sections of various lengths, several different widths of transverse joint opening were provided. The shorter sections were separated by conventional dowel-type joints having widths of either ¾ or 1 inch. A joint of a type similar to that frequently used at bridge approaches, and designed to permit a 1½-inch movement in each direction, was placed between intermediate-length sections; whereas for the longer sections provision was made for approximately twice this amount of movement by means of a pair of the bridge-type joints spaced 10 feet apart.

In addition to the regular sections of the experimental pavement—that is, the sections containing continuously bonded steel—four special sections each 500 feet long were included. In these, weakened-plane joints were spaced at 10-foot intervals and the bond between the longitudinal steel and the concrete was broken purposely for a distance of 18 inches on each side of each transverse joint.

[1] *Experiments with continuous reinforcement in concrete pavements*, by E. C. Sutherland and S. W. Benham; PUBLIC ROADS, vol. 20, No. 11, Jan. 1940; *Progress in experiments with continuous reinforcement in concrete pavements*, by H. D. Cashell and S. W. Benham; PUBLIC ROADS, vol. 22, No. 3, May 1941; and *Experiments with continuous reinforcement in concrete pavement—A five-year history*, by H. D. Cashell and S. W. Benham; Proceedings of the Highway Research Board, vol. 23, 1943 (condensed).

CONCLUSIONS

In this report the performance of the continuously reinforced sections is traced through the first 10 years of pavement life. The following statements give what appear to be the most significant conclusions to be drawn from the results of this investigation.

Changes in Elevation and Length

1. Changes in pavement elevation were generally small and nonuniform, the lack of uniformity becoming progressively more pronounced, especially during the first 5 years of service. The effect of these nonuniform elevation changes was not apparent in either the length changes or the crack patterns of the sections; but, as would be expected, was reflected in the riding quality of the pavement.

2. Because of the wide range in section lengths an opportunity was afforded to study the effect of subgrade resistance as related to slab movement. The most important conclusions are: (*a*) excepting the very short sections, the daily and annual changes in section lengths are not directly proportional to length of section; (*b*) the magnitude of the restraint offered by the subgrade is a function of the time during which a given temperature or moisture change in the pavement takes place; (*c*) for subgrade soil of the type on which the experimental pavement was constructed, it is estimated that, during the relatively rapid daily length change, the central region of sections greater than approximately 800 feet will be in a state of complete restraint; whereas for the slowly developed annual length change, the central region of sections somewhat greater in length than the longest section (1,310 feet) of this investigation will be completely restrained; and (*d*) for sections of lengths included in this investigation, the data suggest that tensile stresses induced by subgrade resistance are probably larger during the fall than at any other period during the year.

3. Length changes of a progressive or permanent nature developed in sections of all lengths. In the short sections containing comparatively few cracks, it appeared that repeated cycles of moisture and temperature were primarily responsible for such changes. In the longer sections, the tendency of the transverse cracks to open progressively a very small amount was an additional factor contributing to permanent increases.

Development of Cracks

4. Transverse cracks in the experimental sections formed essentially at right angles to the axis of the pavement. The surface widths of these cracks because of slight raveling became, in time, much greater than their real widths. For a given computed maximum steel stress both the surface widths and the real widths of the cracks increased approximately directly with a decrease in the percentage of longitudinal reinforcement. In the heavily traveled lane, after 10 years of service, the average values of the real width of the cracks (obtained in the fall of the year in the central region of the longest section for each percentage of reinforcing steel) ranged from 0.004 inch for the section with 1.82 percent steel to 0.011 inch for the section with 0.45 percent steel. Likewise, the average surface widths of the same cracks ranged from 0.05 to 0.10 inch.

5. The rate of crack development was most pronounced during the early life of the pavement, the greatest rate being between spring and fall of the first year after construction. On the basis of all transverse cracks that developed during the 10-year service period, 67 percent had appeared by the end of the first year. Very few cracks formed during the winter months, indicating that nonuniform changes in pavement elevation caused by frost penetration had little influence upon cracking; and, also, that tensile stresses originating from subgrade resistance were no greater during winter periods than during the fall periods.

6. The study of crack development indicated that the average interval between transverse cracks (average slab length) increased with an increase in section length until a peak value was reached, beyond which there was a rapid decrease in average slab length that became more gradual and finally approached a constant value for the longer sections. If one were interested only in the minimum number of transverse cracks and joints, the data suggest that, in reinforced concrete pavements, the joints should be spaced at approximately 100-foot intervals. It must be kept in mind, however, that this investigation most conclusively shows that the character and not the number of cracks is of the greater importance. In the longer and consequently more heavily reinforced sections, many fine cracks have developed at frequent intervals but these sections have continued to be strong, durable structural units after 10 years of heavy traffic service.

7. The frequency of cracking increased from a minimum value at the end of a section to a maximum value in the central area. For some distance, beginning at the end of the longer sections, the frequency of cracking increased directly with increase in distance. The maximum values of crack frequency, as found in the central area of the sections, increased progressively with increase in section length. It seems reasonable to assume that such values would continue to increase until the sections are long enough to develop complete restraint to slab movement. The 10-year data suggest that, for the conditions obtaining in this investigation, the crack interval in the region of complete restraint might be expected to be approximately 2.0 to 2.5 feet provided, of course, that the reinforcement was adequate.

8. Repetition of traffic loads had only a slight influence on the development of transverse cracks. At the end of 10 years 53 percent of the transverse cracks present in both lanes of the pavement formed in the heavily traveled right-hand lane. However, the greater volume of traffic using the right-hand lane produced more raveling and other superficial damage to the edges of the cracks than the lighter traffic on the left or passing lane, the average surface width of cracks in the heavily traveled lane being approximately three times that of the cracks in the passing lane. Traffic had some effect, also, on the real widths of the cracks, this being less pronounced than in the case of the surface widths.

Effect of Reinforcement

9. All sections were so conservatively designed that the limiting length of section for each percentage of longitudinal steel was not determined.

10. Longitudinal steel reinforcement, within the range of the computed maximum stress values of this investigation, held closed all cracks excepting those in the sections reinforced with the 32-pound wire fabric. In several cases the steel crossing the cracks that formed in the sections containing this light fabric broke, probably from shearing forces. It is indicated that wire fabric as light as 32 pounds per 100 square feet should be used with caution as reinforcement in concrete pavements.

Installing one-fourth inch diameter longitudinal bar reinforcing steel.

11. The presence of the heavy longitudinal bar reinforcement was not in any way detrimental to the condition of the concrete in the pavement, as attested by the complete absence of longitudinal cracking above such bars and by the continued durability of the concrete. In fact, the manner in which the steel held closed all cracks, especially those in the heavily reinforced sections, is believed to be conducive to distributed interfacial pressure and should minimize damage of the concrete from concentrated pressure such as sometimes develops at cracks in plain concrete pavements.

12. The type of reinforcement had only a slight effect on the observed length changes of the sections or on the frequency of cracking in the central portion of long sections; but, for some unknown reason, the reinforcement type seems to have had considerable influence on the average interval between cracks in sections of 300 feet or less in length. On the other hand, the working stresses within the range of the computed maximum values exercised no significant control over the length changes and crack patterns of the sections, probably because of the conservative design assumptions.

13. Heavy reinforcement caused the length changes and crack patterns of the sections to be quite symmetrical about the center of each section, thus indicating a structure of predictable behavior.

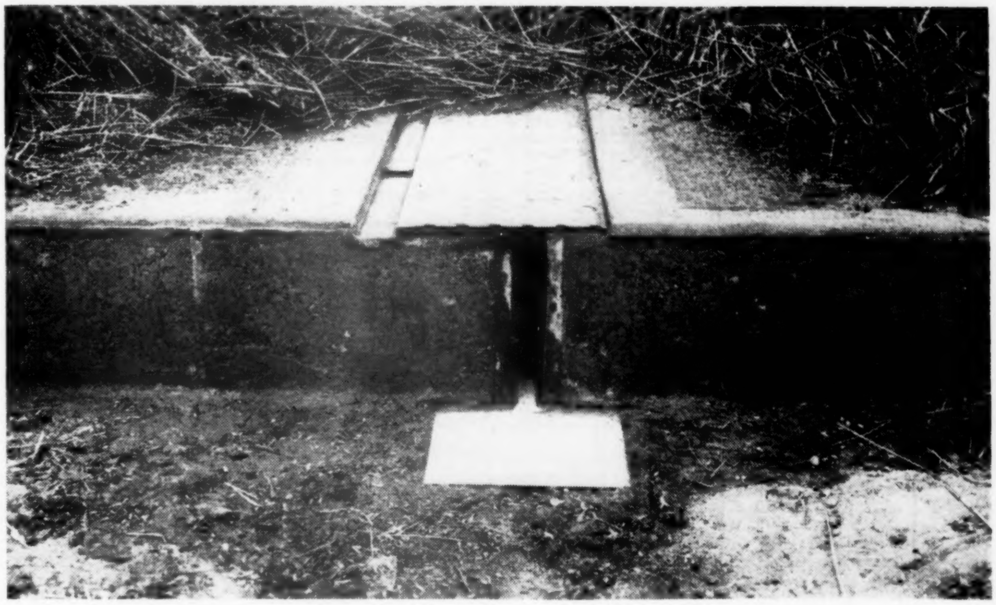
The Special Sections

14. In the four special 500-foot sections, containing warping joints at 10-foot intervals, certain inherent weaknesses developed during the 10 years of traffic service. These weaknesses point out, first, the necessity of providing the warping joints with load-transfer or shear units; and, second, the need of a heavier wire fabric than 45 pounds per 100 square feet. In the halves of the sections provided with shear bars and containing the 91-pound fabric, the steel did not break or inelastically elongate and the pavement remained structurally intact, the riding quality of these halves being nearly the same as that of sections reinforced with continuously bonded steel.

15. During periods of contraction, the continuous reinforcement in the 500-foot special sections exercised considerable control over the length changes of the section as a whole. In spite of the continuity of the reinforcement, however, the warping joints opened progressively with time. This behavior is definitely undesirable since a residual opening of the joints would dissipate all or part of the elastic elongation of the 36 inches of unbonded steel and, in time, may cause failure of the relatively lightweight reinforcement. A corrective measure for the preceding condition would be to decrease the amount of available expansion space.

Pumping and Pavement Smoothness

16. Pumping developed at many expansion joints, but, with two exceptions, was completely absent at the vast number of transverse cracks. This is evidence of the effectiveness of the reinforcing steel in holding tightly together the segments of the sections, thus reducing slab deflections and minimizing the passage of free water to the subgrade soil. The absence of pumping at the submerged-type warping joints of the special sections indicated that the copper seals which enveloped the bottom parting strips prevented the leakage of free water to the subgrade soil.



The bridge-type joint.

17. Relative roughness determinations of the regular sections showed that their surfaces were very smooth initially, and at present are no rougher than some concrete pavements as constructed. The many fine cracks that formed in the long sections have not affected the riding quality of the pavement. The sections containing warping joints at 10-foot intervals were as smooth initially as the regular sections indicating that, with proper care during installation and finishing, closely-spaced warping joints need not affect the initial riding quality of concrete pavements. However, where certain weaknesses have developed, such as faulting of the joints, these special sections have become much rougher than the regular sections.

Economic Benefits

The performance of the Indiana experimental sections has indicated certain economic benefits to be derived from long, continuously reinforced pavement of the type included in this investigation, namely: (1) the fine cracks, even though frequent, ravel only slightly with traffic and exposure, a condition that may be considered superficial and one that will require no maintenance; (2) except in localized areas of extremely poor subgrade, pumping will not develop, thus minimizing the need for base courses or other expensive subgrade treatments; and (3) the riding quality of the pavement might be expected to remain excellent and the pavement itself would be protected from damaging impact forces such as frequently develop at faulted joints and cracks.

More recently, other researches relating to the use of continuously bonded longitudinal reinforcement have been inaugurated.[2] The experimental sections in these pavements are all longer than the longest section of the Stilesville experimental pavement and are less heavily reinforced. Sections of various thicknesses, reinforced with various percentages of longitudinal steel, and constructed both with and without subbases, are included. While the pavements have not been in service long enough to permit conclusions to be drawn, it seems probable that eventually they will provide considerable additional data on the relative economies of continuously reinforced pavements and those which are designed as a series of comparatively short, independent slabs.

Thus, it is possible that, when all factors are considered, a concrete pavement without joints and reinforced with continuous bonded steel in sufficient amount to resist all stresses safely and to hold all cracks closed may, in many cases, cost no more than current designs of concrete pavement which include greater slab thickness, lighter reinforcement, transverse joints, and subgrade treatment.

SCOPE OF DISCUSSION

The discussion of the 10-year performance of the pavement is presented in six parts, as follows: (1) Periodic elevation changes of the regular sections; (2) daily, annual, and progressive changes in the length of the regular sections; (3) development, distribution, and present condition of cracks in the regular sections; (4) behavior of the 500-foot special sections; (5) occurrence of pumping; and (6) smoothness of the pavement. Pertinent data on the pavement sections and the reinforcing steel in them are given in table 1.

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[2] *Continuously reinforced concrete pavements without joints*, by W. R. Woolley; *Preliminary report on current experiment with continuous reinforcement in New Jersey*, by W. Van Breemen; and *An experimental continuously reinforced concrete pavement in Illinois*, by H. W. Russell and J. D. Lindsay; Proceedings of the Highway Research Board, vol. 27, 1947.

Table 1.—*Details of reinforcement in the experimental pavement*

Number of sections for each length [1]	Length of each section [2]	Calculated maximum stress in steel	Weight of reinforcement	Reinforcement size and spacing: Longitudinal, Diameter [3]	Longitudinal, Spacing, c. to c.	Transverse, Diameter [3]	Transverse, Spacing, c. to c.	Percentage of longitudinal steel [4]	Average tensile strength of longitudinal steel: Yield point	Ultimate
	Feet	*Lb. per sq. in.*	*Lb. per 100 sq. ft.*		*Inches*		*Inches*	*Percent*	*Lb. per sq. in.*	*Lb. per sq. in.*
RAIL-STEEL BARS (DEFORMED)										
2	600	25,000		1-inch	6	½-inch	24	1.82	63,300	113,200
	840	35,000								
	1,080	45,000								
	1,320	55,000								
4	340	25,000		¾-inch	6	½-inch	24	1.02	64,400	113,300
	470	35,000								
	610	45,000								
	740	55,000								
4	150	25,000		½-inch	6	½-inch	24	.45	68,800	115,300
	210	35,000								
	270	45,000								
	330	55,000								
6	80	25,000		⅜-inch	6	⅜-inch	24	.26	66,700	93,600
	120	35,000								
	150	45,000								
	180	55,000								
6	40	25,000		¼-inch	6	¼-inch	12	.11	60,300	84,600
	50	35,000								
	60	45,000								
	80	55,000								
BILLET-STEEL BARS (DEFORMED), INTERMEDIATE GRADE										
2	360	15,000		1-inch	6	½-inch	24	1.82	46,900	78,000
	600	25,000								
	840	35,000								
	1,080	45,000								
4	200	15,000		¾-inch	6	½-inch	24	1.02	49,100	78,500
	340	25,000								
	470	35,000								
	610	45,000								
4	90	15,000		½-inch	6	½-inch	24	.45	51,400	78,600
	150	25,000								
	210	35,000								
	270	45,000								
6	50	15,000		⅜-inch	6	⅜-inch	24	.26	55,500	81,900
	80	25,000								
	120	35,000								
	150	45,000								
6	20	15,000		¼-inch	6	¼-inch	12	.11	56,900	77,300
	40	25,000								
	50	35,000								
	60	45,000								
WIRE FABRIC (COLD-DRAWN WIRES)										
6	140	25,000	149	No. 0000	4	No. 3	12	.42	------	81,800
	190	35,000								
	250	45,000								
	310	55,000								
6	90	25,000	107	No. 0000	6	No. 3	12	.28	------	80,300
	130	35,000								
	170	45,000								
	200	55,000								
6	80	25,000	91	No. 000	6	No. 4	12	.24	------	89,100
	110	35,000								
	140	45,000								
	170	55,000								
6	60	25,000	65	No. 0	6	No. 6	12	.17	------	83,700
	80	35,000								
	100	45,000								
	120	55,000								
6	30	25,000	45	No. 3	6	No. 6	12	.11	------	81,000
	50	35,000								
	60	45,000								
	80	55,000								
6	20	25,000	32	No. 6	6	No. 6	12	.07	------	88,700
	30	35,000								
	40	45,000								
	50	55,000								

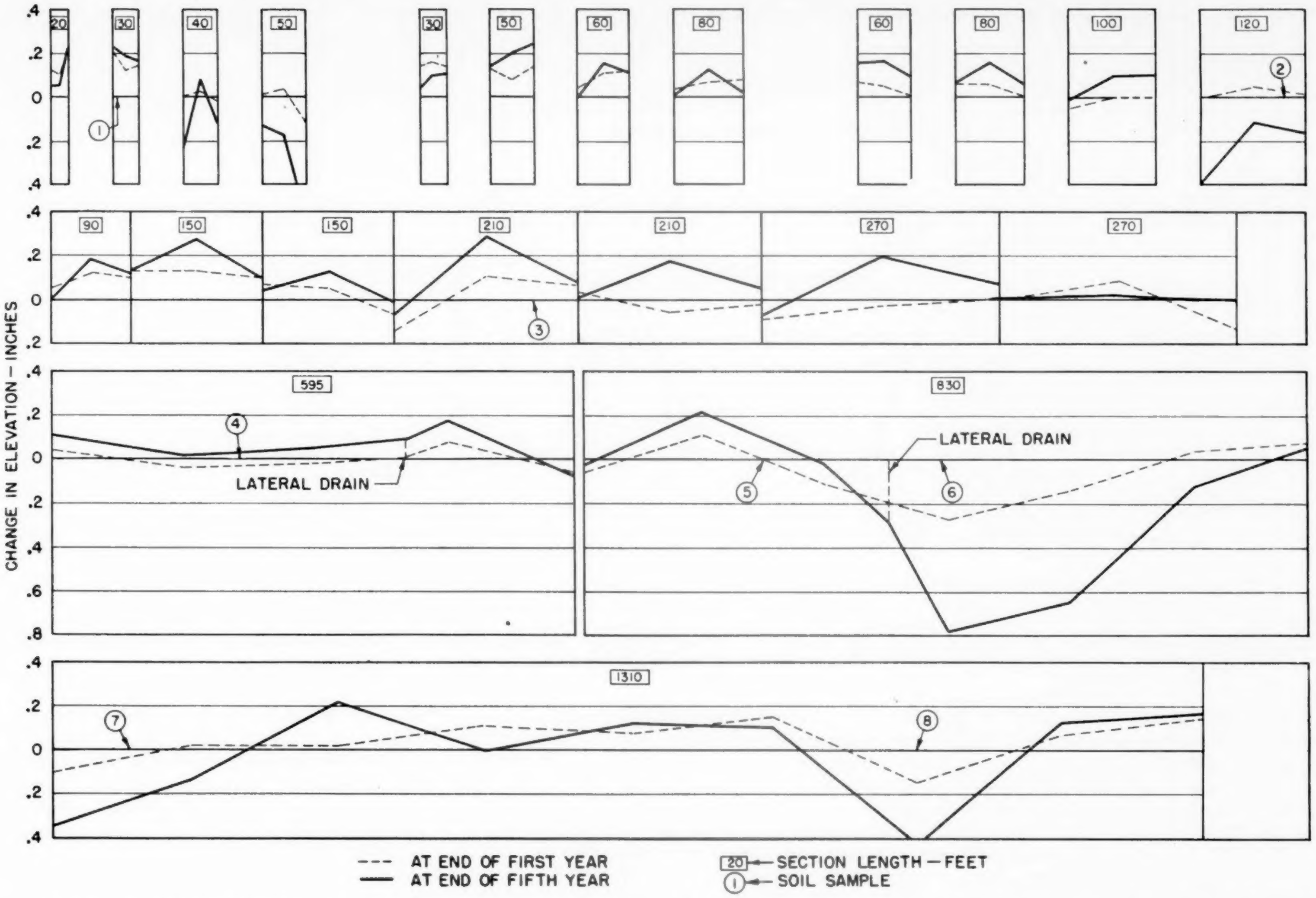
[1] The term "section" as used in this report refers to a lane or 10-foot width of pavement; thus the number "2" indicates a pair of sections, one being on each side of the center joint.
[2] The lengths of the longer sections are nominal lengths and may be either 5 or 10 feet greater than the actual length in cases where a pair of bridge-type joints were installed.
[3] The rail-steel and billet-steel reinforcement were round bars. The diameters of the wires in the fabric were as follows: No. 0000, 0.3938 inch; No. 000, 0.3625 inch; No. 0, 0.3065 inch; No 3, 0.2437 inch; No. 4, 0.2253 inch; No. 6, 0.1920 inch.
[4] Cross-sectional area of the longitudinal steel expressed as a percentage of the cross-sectional area of the concrete slab.

Part 1.—PERIODIC ELEVATION CHANGES

Three sets of precise elevation measurements have been made over the entire length of the experimental pavement: In the late fall of 1938 or shortly after construction of the sections; in the fall of 1939 or approximately 1 year after construction; and in the severe winter of 1939–40 when the frost had penetrated the ground to a depth of about 20 inches. In addition, during the first 5 years, elevation measurements were made at more frequent intervals over selected sections. All such measurements were made on reference points installed in the right-hand or heavily traveled lane of the pavement.

At the end of the first year the majority of the elevation changes were very small. Specifically, only 7 percent of the 487 midlane locations at which measurements were made showed a change in elevation greater than one-fourth inch when compared with the base elevations established soon after construction.

During the peak of the severe winter of 1939–40 increases in elevation were generally within the range of 0.2 to 1.0 inch as compared



SOIL SAMPLE NO.	1	2	3	4	5	6	7	8
SILT—PERCENT	38	43	60	58	63	39	46	43
CLAY—PERCENT	16	18	18	18	13	12	19	18
LIQUID LIMIT	32	34	36	37	43	28	35	29
PLASTICITY INDEX	14	11	12	12	19	11	14	11

SOIL SAMPLE NO.	1	2	3	4	5	6	7	8
	MOISTURE CONTENT—PERCENT							
0–3 INCH DEPTH	16.0	11.0	13.6	11.5	12.9	19.2	16.8	9.8
3–12 INCH DEPTH	18.0	13.0	18.0	14.6	15.8	13.5	18.7	15.7
12–24 INCH DEPTH	18.0	19.0	18.0	13.1	20.9	13.5	19.6	12.8

Figure 1.—Changes in elevation of selected sections at the end of the first and fifth years of pavement life, and physical characteristics of the subgrade soil at the time the pavement was placed.

with the elevations determined the previous fall. It was observed also, in most instances, that heaving was greater at the expansion joints than at points elsewhere in the sections, averaging 0.47 inch at 151 expansion joints and 0.33 inch at 185 points elsewhere in the sections. These data emphasize the importance of tightly sealed joints in pavements exposed to freezing conditions.

Examples of typical changes in pavement elevation are shown in figures 1 and 2.

The elevation changes in figure 1 are those observed on selected sections in the fall at the end of the first and fifth years of pavement life, using as a base the elevations established shortly after construction. Subgrade soil data applicable at the time the pavement was placed are also given in this figure.

The data of figure 1 indicate that the elevation changes at the end of 5 years were appreciably greater in magnitude and were less uniform than at the end of the first year. The greatest change was a settlement of 0.8 inch near the center of the 830-foot section. However, measurements taken in this area at the end of 10 years showed virtually no change with respect to the 5-year profile.

Figure 2 shows the changes in elevation of a composite section, this figure being prepared from measurements made from time to time at the centers and at the ends of 24 representative sections, using as a base the elevations established soon after construction. Thus, the elevation changes shown for the center and for the end of the composite section are, respectively, the average changes in elevation of the 24 centers and of the 48 ends of the representative sections.

The positions of the center and end of the composite section indicate that the pavement as a whole raised slightly with respect to the basic elevation; and that, excepting in the severe winter of 1939–40, the changes in elevation were greater at the center than at the end, suggesting a permanent downward warping at the end of the composite section with respect to its center. It is believed that soil displacement resulting from pumping or consolidation of the subgrade at the joints may be primarily responsible for this condition of distortion.

Part 2.—CHANGES IN LENGTH

Daily, annual, and progressive changes in length were measured at the ends of a number of representative sections. These length changes were carefully determined either by measurement to fixed reference points located at the ends of a section or by measurements across joints between sections of equal length. Cross-joint measurements give the width changes of joints which, when determined for a joint separating sections of equal length, should approximate the total length change of one of the joining sections. All of these measurements were from points installed in

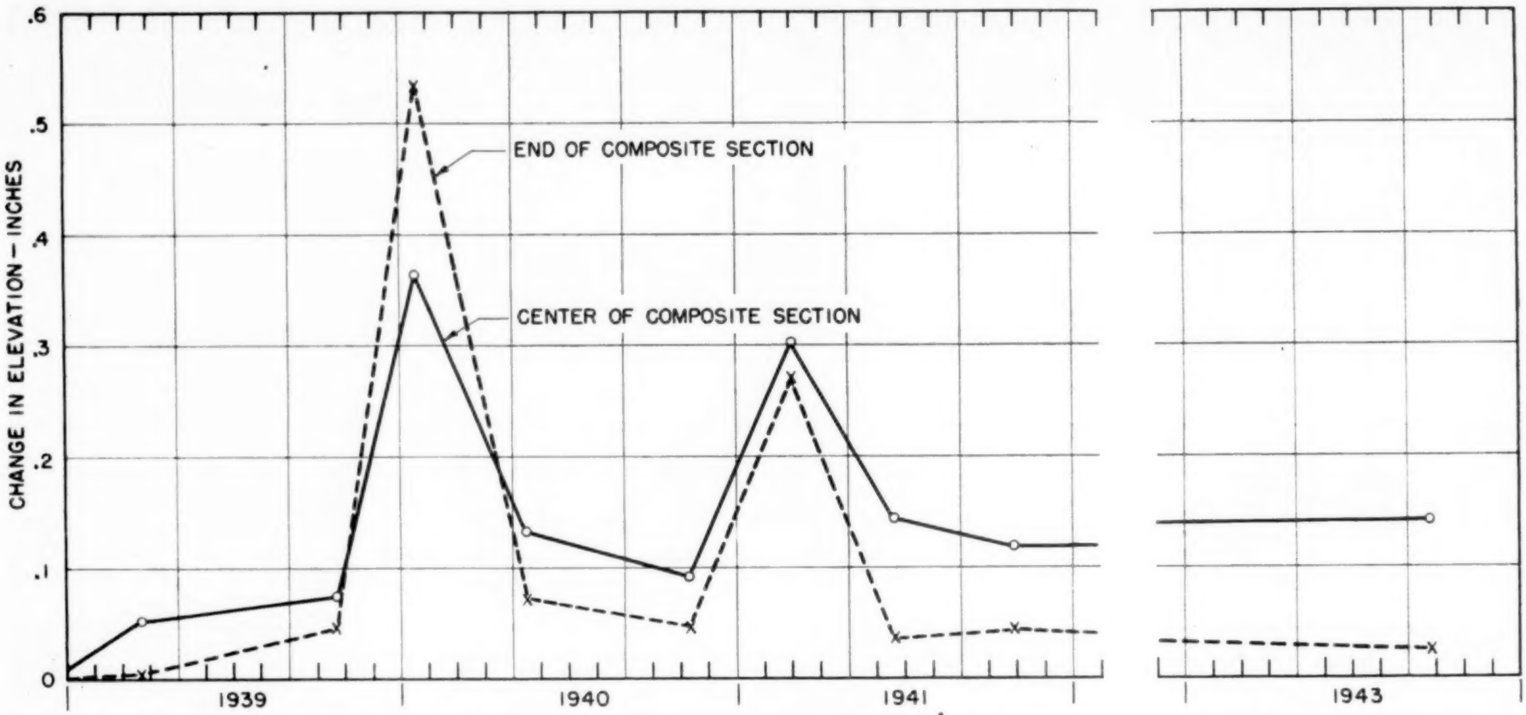


Figure 2.—Changes in elevation of the center and end of a composite section (average of 24 representative sections); base measurements obtained in December 1938.

the surface of the right-hand or heavily traveled lane of the pavement.

It will be recalled that all transverse joints were designed to care for a reasonable amount of slab expansion and, as far as can be determined, the observed length changes of the sections were unaffected by restraint at the joints during the first 5 years of pavement life. Considerable care was exercised during construction in correctly alining the round steel dowels used in the joints which separate the shorter sections, so that restraint from this source was reduced to a minimum. Over a period of time, however, the joints, especially the bridge-type joints, gradually became filled with soil and with bituminous material used for their maintenance so, at present, the movements of several of the longer sections may be restrained to some extent during periods of maximum expansion.

For a given temperature or moisture change in the concrete, there should be a proportionate change in section length provided the section remained structurally intact and was not restrained in any manner. Actually, however, the length changes of the sections of this study are affected by any restraint that may develop at transverse joints and by such factors as subgrade resistance to slab movement, differences in the thermal coefficients of steel and concrete, moisture changes of the concrete and not of the steel, and changes in width of existing transverse cracks. Just how much influence each of these factors exerts cannot be determined, but it is believed that the subgrade resistance is the most important. Since the thermal coefficients of steel and concrete are nearly the same (that for steel being somewhat the greater) it seems reasonable to expect that this factor would have little effect on the length changes of the sections, particularly during expansion. As for transverse cracking, it will be shown later in the report that very few cracks occurred in sections having lengths less than 120 feet and the cracks that developed in the longer, more heavily reinforced sections were extremely fine. Thus, for either a daily or annual time period, the cumulative change in width of all transverse cracks in a given section should be small compared to the overall length change of the section.

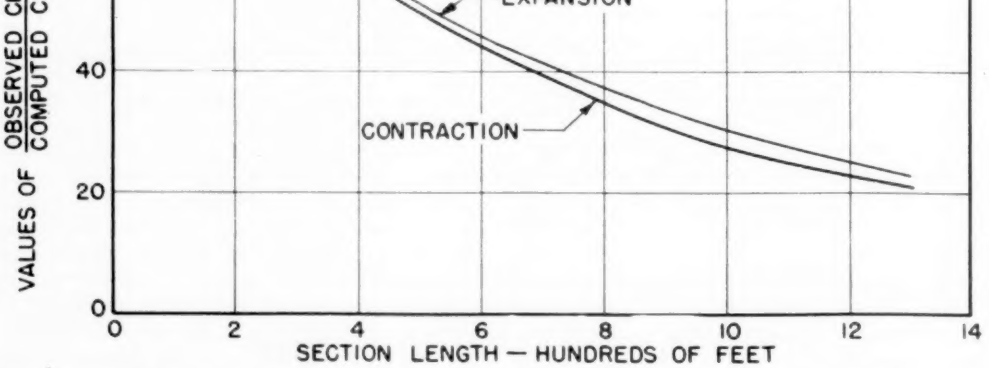


Figure 3.—Observed daily changes in section lengths expressed as percentages of the computed changes in length of equivalent unrestrained sections.

Daily Length Changes

In considering the daily length changes it will be recalled that all measurements at the joints were made at the level of the upper surface of the pavement and were, therefore, affected in some degree by changes in the condition of warping at the ends of the slabs. In this investigation it was found that the corrections for daily warping ranged from 0.003 to 0.005 inch for days when a large temperature change occurred in the pavement. These corrections were applied only to sections having lengths of 60 feet or less because in the longer sections the magnitude

of the length changes was such as to make the correction unimportant.

In figure 4 are shown the relations between section length and change in section length as found for a daily mid-depth pavement temperature drop of 24° F. and a daily mid-depth pavement temperature rise of 30° F. The data for this figure were obtained from 64 sections that cover the range of section lengths for all percentages of longitudinal steel included in each of the three types of reinforcement. The slopes of the straight lines shown in the figure were computed from the change in length of 19 uncracked sections, 20 to 60 feet long, and thus represent, for the temperature changes mentioned, the rates of length change for short sections that are comparatively free to expand and contract. From these the coefficients of daily length change for short sections, which should approximate the thermal coefficient of the concrete, are readily obtained.

It is apparent from this figure that sections up to approximately 75 feet in length move with as much freedom as the very short sections. The change in length of sections greater than about 75 feet is restrained by subgrade resistance and perhaps other factors and this restraint, the effect of which is shown as departure from the slope line established by the short sections, increases rapidly as the sections become longer. After a section length of about 800 feet is reached, the curves level off, indicating that the maximum restraint has been developed and that sections whose lengths are greater than this may be expected to show total length changes of equal magnitude. This suggests that the central portion of sections greater than 800 feet will be completely restrained during quick changes in average concrete temperature. Applied to the 1,310-foot section, this would mean that the central 500± feet of this section did not move during the daily temperature changes for which data are shown in figure 4.

The great influence of subgrade resistance on the relatively rapid daily change in length of the sections is clearly indicated in figure 3. This figure, prepared from the curves of figure 4, shows the observed change in length of a section expressed as a percentage of the change in length of an unrestrained section of the same length. The length changes of the unrestrained sections were calculated from the data obtained from the very short sections. Reduced to this basis, the observed daily length changes were about the same for both the expansion and contraction cycles.

In the discussion of figure 4 it was suggested that the central portion of long sections may be completely restrained during quick changes in pavement temperature. This seems to be confirmed by the data given in figure 5, data which show the longitudinal movements observed at the center, quarter-points, and ends of a 1,310-foot section for a daily mid-depth temperature drop of 19° F. and a rise of 25° F. The values shown for the quarter-point and for the end are in each case the average value obtained from measurements at both quarter-points and at both ends of the section.

These data indicate that during contraction

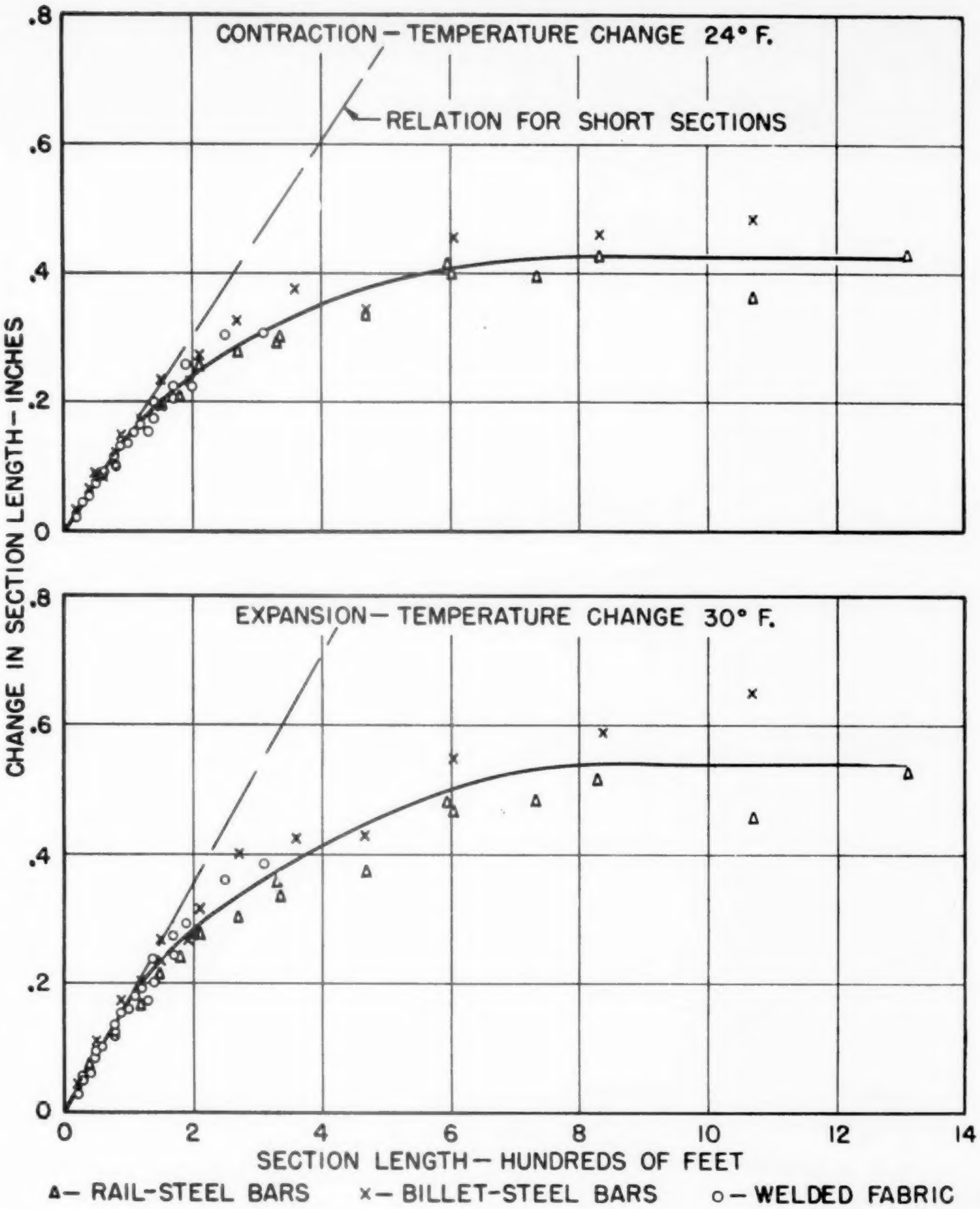


Figure 4.—Relation between section length and daily change in length.

the movement at the ends of the 1,310-foot section was about 20 percent and that at the quarter-points about 1 percent of the movement which would be found in an unrestrained section of the same length. Corresponding values for expansion were about 20 and 2 percent. It will be noted that between 500 and 550 feet of the central portion of the 1,310-foot section did not move during the daily cycle, this length being comparable to the 500±-foot length that was estimated from the results of figure 4.

The manner in which certain sections respond to a daily rise in pavement temperature is shown in figure 6. The basic measurements for this figure were obtained in the early morning of a summer day and subsequent measurements were made at intervals until late afternoon.

In the case of both the 470- and 1,310-foot sections the relation between increase in temperature and change in over-all section length remained linear until a total elongation of approximately 0.2 inch was attained, after which the rate of length change increased progressively with temperature, being more pronounced for the longer section. For example, during the 28° F. temperature rise shown in figure 6, the 1,310-foot section moved 0.19 inch for the first 14° temperature increase and 0.27 inch for the second 14° increase. Since the movements of the sections are intimately related to the restraint offered by the subgrade, the increase in the rate of length change of the long sections suggests that, after a certain amount of slab displacement, the total or accumulated subgrade resistance continues to increase, but at a progressively decreasing rate.

Daily changes in length of a limited number of sections were observed each summer over a 4-year period on days when a large temperature change occurred in the pavement. This 4-year period extended from the second through the fifth year of pavement life and, therefore, it is believed that the movements

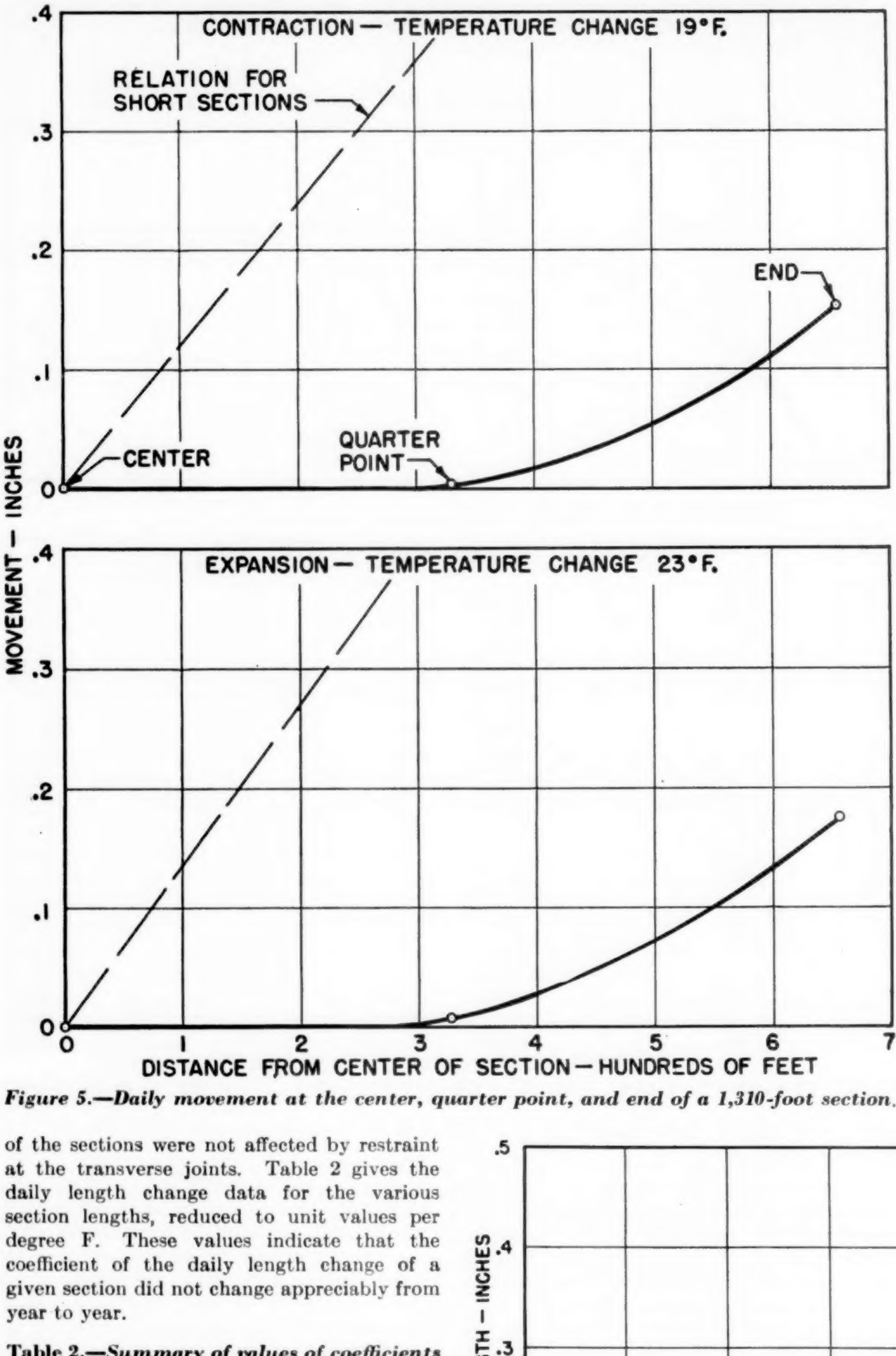


Figure 5.—Daily movement at the center, quarter point, and end of a 1,310-foot section.

of the sections were not affected by restraint at the transverse joints. Table 2 gives the daily length change data for the various section lengths, reduced to unit values per degree F. These values indicate that the coefficient of the daily length change of a given section did not change appreciably from year to year.

Table 2.—*Summary of values of coefficients of daily length change (based on changes in over-all section length)*

Section length	Unit change in section length per degree F. $\times 10^{-7}$					
	July 1940 A. M. to P. M.	June 1941		June 1942		July 1943 A. M. to P. M.
		P. M. to A. M.	A. M. to P. M.	P. M. to A. M.	A. M. to P. M.	
Feet						
20	49	49	49	41	52	---
150	---	46	43	44	41	---
335	29	32	29	---	---	---
470	---	25	25	22	24	---
600	21	24	24	21	23	---
1,070	13	13	14	---	---	---
1,310	9	11	11	8	10	10

The average of the five values obtained from the observed daily length change for the 20-foot section (table 2) is 0.0000048 per degree F. Because the 20-foot section is free to expand and contract when subjected to a temperature change, this value should approximate the thermal coefficient of the concrete. As a matter of supporting data the slopes established by the short sections, as shown in figure 4, when divided by the temperature change of the concrete, give coefficient values of 0.0000053 per degree F. for contraction and 0.0000049 per degree F. for expansion.

Annual Length Changes

Figure 7 contains the annual length change data for the various sections for the first, second, third, and fifth years of the life of the pavement. The annual change in length of a section was computed from data obtained in the morning of a midwinter day and in the afternoon of a midsummer day and, consequently, includes the length change that occurred between the morning of a winter day and the morning of a summer day plus the daily length change that occurred between the morning and afternoon of the aforementioned summer day. Since an effort was made to obtain these data during the coldest period of winter and the hottest period of summer, the length changes shown are approximately the maximum for the annual cycle. The slopes of the straight lines represent respective annual relations determined from 19 uncracked short sections.

The type of reinforcement used in the various sections is denoted by symbol. There appears to be some tendency for sections containing billet-steel bars to develop slightly greater annual length changes than equivalent-length sections reinforced with rail-steel bars. This same tendency was noted in the daily expansion and contraction data of figure 4. The cause for this difference is not known.

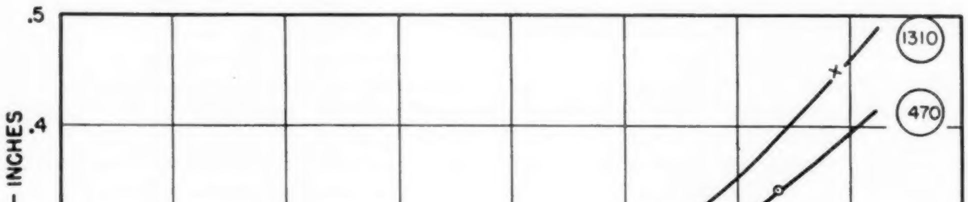


Figure 6.—Effect of a daily rise of the mean pavement temperature on the change in length of several sections (figures in circles indicate the length of sections in feet).

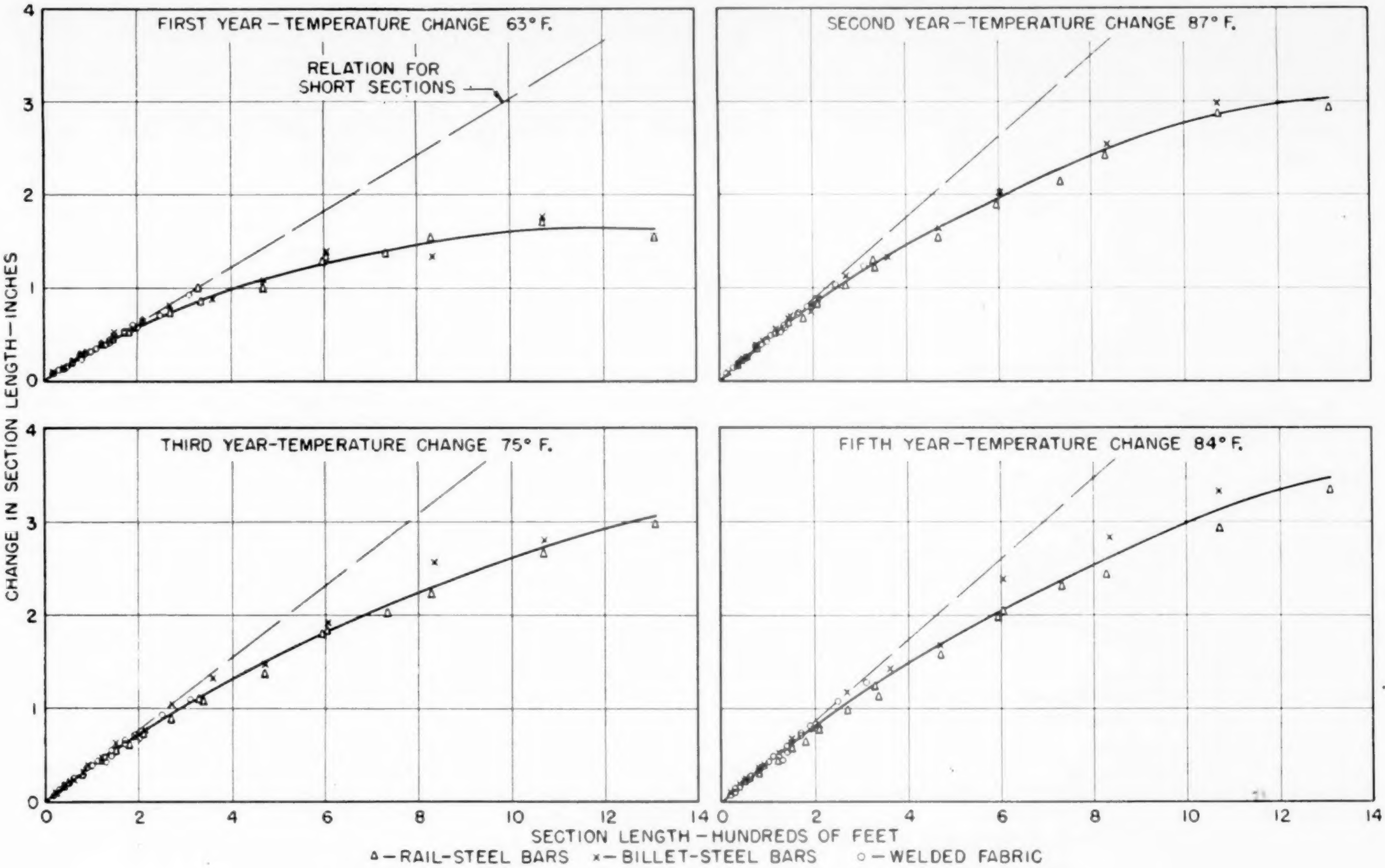


Figure 7.—Relation between section length and annual change in length.

It is indicated by the four curves of figure 7 that sections up to approximately 150 feet in length move with as much freedom during an annual cycle as do the very short sections. The length changes of sections greater than 150 feet are restrained by the subgrade, however, and this restraint increases progressively with increase in section length. The data of figure 4 indicated that daily restraint to free movement was first noticeable in sections about 75 feet long. It should be remembered that the annual length change data considered above include the effects of one daily cycle also. A probable explanation for the observed difference, just mentioned, is that, under the slowly developed temperature rise from winter to summer, sections up to at least 150 feet in length moved freely because they encountered less restraint from the subgrade than obtains during the more rapid daily cycle of length change. Hence, the small amount of daily restraint to free movement of sections between 75 and 150 feet in length, as shown in figure 4, while present is not apparent in the curves of figure 7.

In connection with this study of the annual length changes of the sections it is of interest to note the symmetry of movement that was found in the long sections. For example, during the fifth annual period, the observed movement at one end of a 1,070-foot section was 1.50 inches while at the other end the movement was 1.42 inches; likewise the movements at the two ends of the 1,310-foot section were 1.62 and 1.72 inches, respectively.

To provide a more easily visualized comparison of the annual length changes of the various sections, figure 8 was developed from figure 7 in the same manner that figure 3 was obtained from figure 4. The curves of figure 8 show not only the magnitude of the restraint that was present in the various sections, but also that the sections expanded more freely

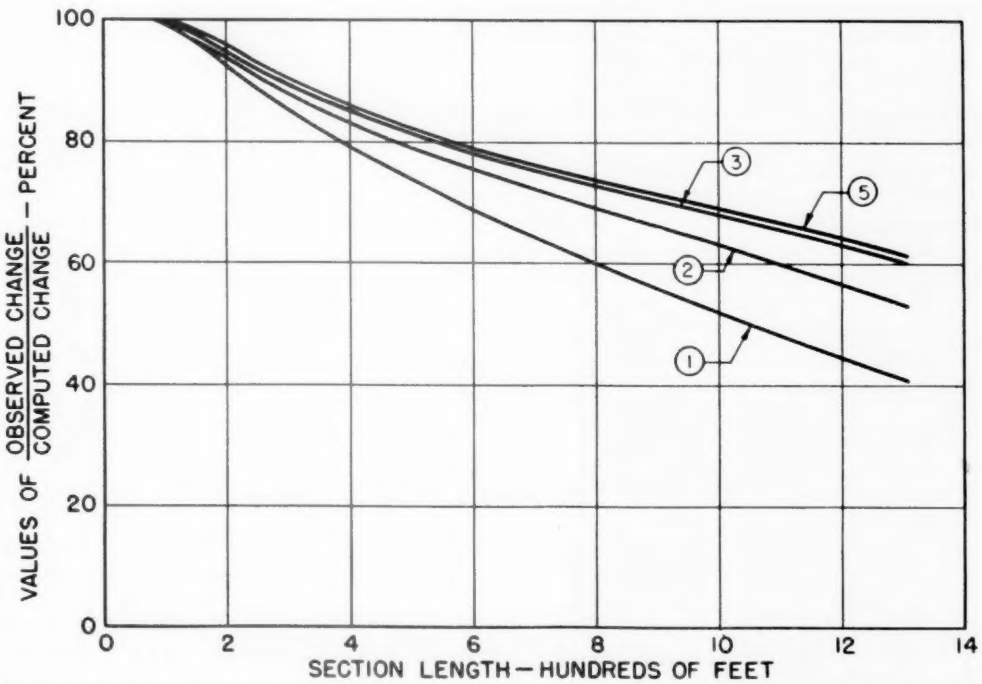


Figure 8.—Observed annual changes in section lengths expressed as percentages of the computed changes in length of equivalent unrestrained sections (figures in circles indicate age of pavement at time of observations).

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progressively for each of the first three annual cycles. Thus, it appears that the sections encountered less subgrade resistance with each successive annual expansion period until, by the end of the third period, a condition of essential stability was reached.

Further evidence is added by the annual movements observed at the quarter-points of the 1,310-foot section. For the first year the annual movement at the quarter-points was about 10 percent of the movement to be expected at the quarter-points of an unrestrained section of equal length. For the second, third, and fifth years the values were respectively 31, 46, and 45 percent.

Greater Freedom of Movement Annually Than Daily

Table 3 shows the annual length changes of selected sections reduced to unit values per degree F. These annual coefficients of length change, although expressed as unit values per degree F., involve temperature, moisture, subgrade resistance, and perhaps other factors. The factor of moisture will be discussed later in the report.

In comparing the coefficient values of the longer sections of table 3 with those of table 2, it is observed that coefficients for an annual expansion period are, in general, much greater than those for a daily expansion period, thus suggesting greater freedom of movement of the sections during an annual period.

This condition is clearly shown, also, by comparing the curves of figure 8 with those of figure 3. For example, during the annual length-change cycle for the 1,310-foot section (fifth year) the observed movement was 62 percent of the theoretical length change for an unrestrained section of equal length; whereas during the daily length change the value was only 22 percent. Hence, it is strongly indicated that the magnitude of the restraint offered by the subgrade is a function of the time during which a given temperature or moisture change in the pavement takes place.

Data lending further support to this observation are given in figure 9. This figure shows the length changes of the sections that occurred between the morning of a day in February when the mid-depth pavement temperature was 32° F. and the morning of a day in late June of the same year when the mid-depth pavement temperature was 77° F. Therefore, the data do not include the effect of a quick daily temperature rise, but rather show only the comparative freedom with which sections of all lengths expanded under a slowly developed temperature rise of 45° F. These measurements were made during the third year of the life of the pavement, at which time stablization of annual movement had developed.

Table 3.—*Summary of values of coefficients of annual length change (based on changes in over-all section length)*

Section length	Unit change in section length per degree F.×10⁻⁷			
	First year	Second year	Third year	Fifth year
Feet				
20	40	42	43	43
150	40	40	41	41
335	34	35	36	36
470	29	32	34	34
600	29	31	34	35
1,070	21	26	28	29
1,310	15	21	25	25

It appears from the cycle of length change shown in figure 9 that sections up to about 900 feet in length expanded as freely as the very short sections. The free movement of sections greater than 900 feet is restrained and, although the sections included in this study are not long enough to warrant definite conclusions, it is indicated by the rapid increase in restraint that the central portion of sections greater in length than approximately 1,700 to 1,800 feet would be in a state of complete restraint during an annual cycle.

The manner in which the sections up to 900 feet in length expanded from February to June indicates that subgrade resistance did not accumulate in these sections and, as a consequence, residual compression was probably absent on the morning of the June day when the pavement temperature was 77° F. Therefore, it seems logical to expect that as summer advances and the mean pavement temperature gradually rises, sections of considerable length, unless restrained at the joints, expand to their annual maximum without developing appreciable residual compression. If this is the case, then it would be expected that in late summer or early fall the comparatively large, sudden drops in temperature would cause comparatively large, direct tensile stresses to be developed in the sections, larger probably than at any other period during the year.

Again confirmatory evidence is supplied by the movements observed at the quarter-points of the 1,310-foot section. During the fifth year of pavement life, a 0.61-inch movement was recorded at the quarter-points of this section when it had expanded to its approximate maximum length for the annual cycle. During the early fall of the same year, however, after the pavement temperature had dropped approximately 50 percent of its winter to summer rise, the contraction of the section was restrained to the extent that the return movement at its quarter-points was only 0.06 inch or about 10 percent of the movement observed at maximum expansion.

In concluding this discussion of the annual length changes of the sections, it is of interest to compare sections in which the three different types of reinforcement were used and, also, those in which the maximum stresses in the longitudinal steel presumably varied considerably, in order to determine the effect of these factors on the relation between section length and annual contraction of the sections. These comparisons are shown in figure 10 for a contraction period since, during such a period, maximum tensile stresses develop in the reinforcement. The length changes given in this figure are the result of a 77° F. fall in temperature that occurred between midsummer and midwinter of the third annual contraction period.

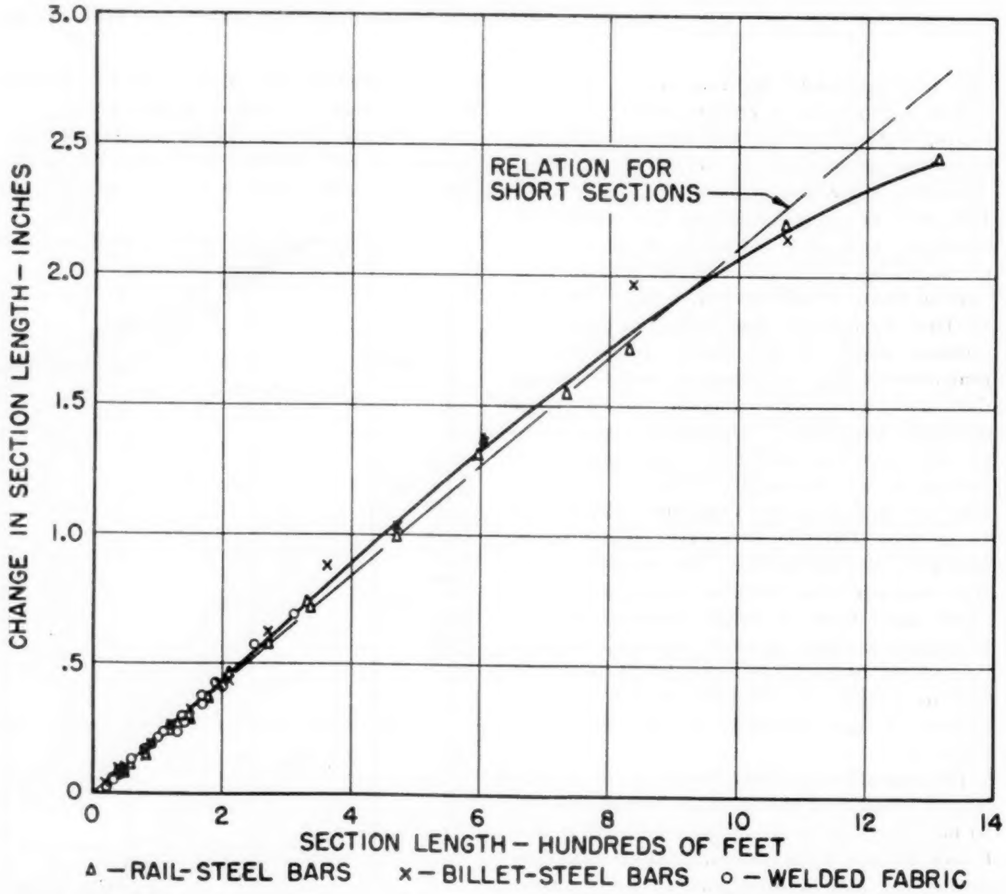

Figure 9.—Relation between section length and change in length from the morning of a winter day to the morning of a summer day of the same year (mean pavement temperature change of 45° F.).

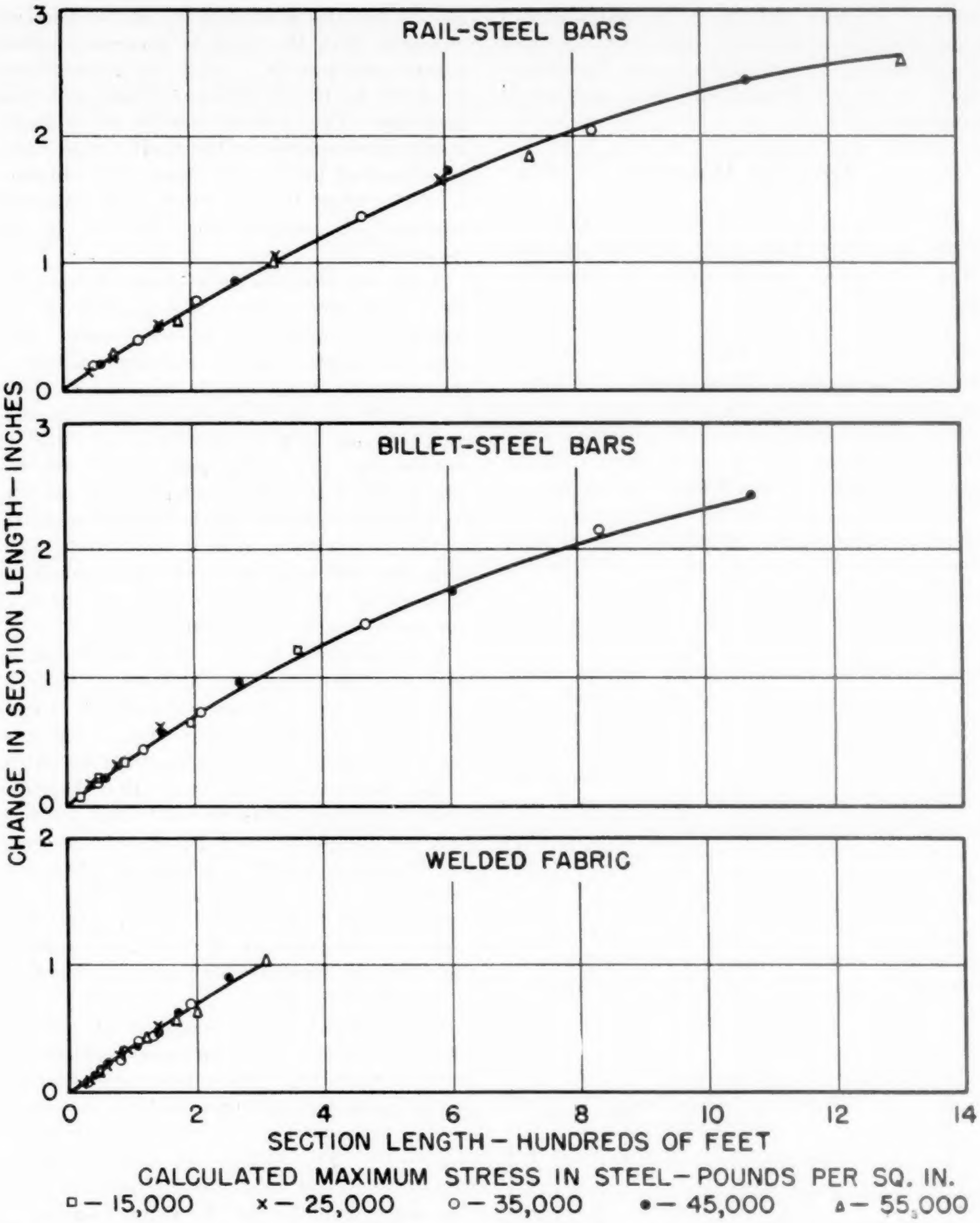

Figure 10.—Effect of type of reinforcement and calculated maximum steel stress on the relation between section length and annual contraction (77° F. temperature drop).

A comparison of the three curves of figure 10 indicates that, for a contraction period, the type of reinforcement had little influence on the observed length changes of the sections. For example, in the case of a 300-foot section the measured length changes for the rail-steel bars, the billet-steel bars, and the welded fabric were 0.92, 0.98, and 1.00 inch respectively.

The maximum steel stresses as calculated for the various sections are denoted, in figure 10, by symbol. For a given section length these stress values may be considered as inverse indices of steel area. The orderly manner in which all points, regardless of symbol, fall on the curves in the figure is evidence that, within the ranges available for comparison in a given section length, the amount of the longitudinal steel exercises no significant control over the length changes. For example, two sections each approximately 600 feet long, reinforced with rail-steel bars, show essentially the same length change although one contains 1.82 percent of longitudinal steel while the other contains but 1.02 percent.

An examination of all of the transverse cracks in the regular sections indicated that, except for 3 in the 24 sections containing the 32-pound wire fabric, all were held closed by the longitudinal steel. The condition of these cracks will be discussed later.

Since after 10 years of heavy-duty service none of the cracks in the regular sections showed evidence of inelastic deformation of the longitudinal steel (except the three just mentioned), it must be concluded that the assumptions used in computing steel stresses in the original design lengths were unduly conservative. How closely the elastic limit of the steel has been approached during this period of service remains unknown.

Progressive Length Changes

To evaluate length changes of a progressive or permanent nature, measurements were made at the ends of a number of selected sections every February and August during the first 9 years of pavement life. The February observations were obtained when the mid-depth slab temperature was approximately 32° F. and the August observations when the mid-depth slab temperature was approximately 92° F. The effect of moisture on the determination of the progressive length changes of the sections was minimized, as much as possible, since there is reason to believe that the moisture content of the pavement remains virtually stable in February and in August. Studies indicate that during these months the absorbed moisture is at the maximum and minimum, respectively, of the annual cycle of moisture change.

The data from the study of progressive or permanent changes in the lengths of the sections are given in figures 11 and 12.

In figure 11 the broken lines show the average February to August and August to February unit length changes of a number of short, uncracked sections, plotted with respect to the initial set of measurements obtained in February of 1939. These sections are 20 to 80 feet in length, comprise 340 feet of pavement in total, and are relatively free to expand and contract. The length changes are expressed as unit values per 60° F. change in slab temperature.

The solid line drawn through the mean

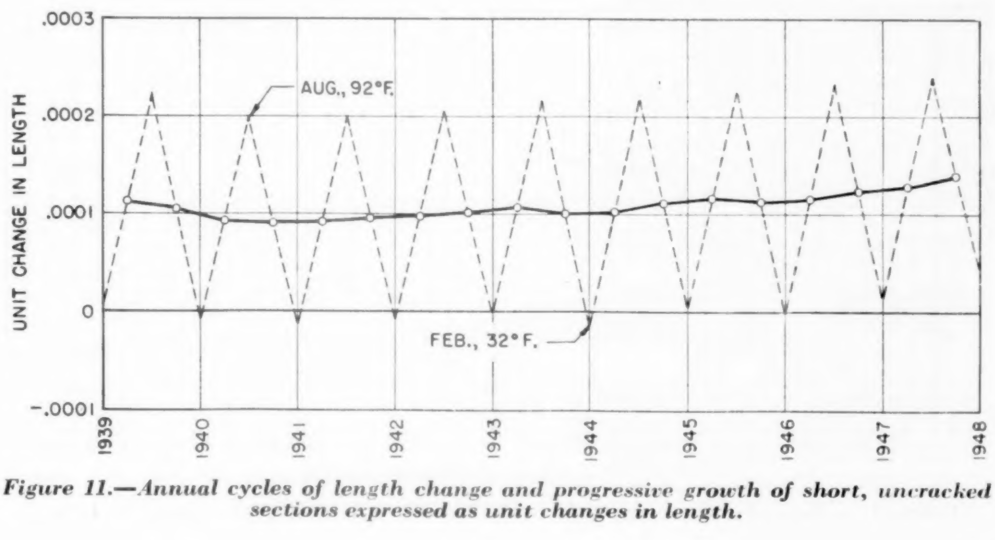

Figure 11.—Annual cycles of length change and progressive growth of short, uncracked sections expressed as unit changes in length.

points of the cyclic variations of figure 11 indicates a progressive permanent growth or increase in the length of the sections, this growth being more pronounced after the fifth year of pavement life. In fact, the growth was so small during the early life of the pavement that in the 5-year report it was stated that "no definite indication of a permanent change in the length of the short sections was observed". Since these short sections are structurally intact there can be little doubt that a permanent increase in length is developing. This appears to be another instance of the tendency of some concretes, at least, to grow when subjected to repeated cycles of temperature and moisture change.

The difference between the high and low points of the mean line of figure 11 represents a permanent unit increase of 0.000048 or an increase of approximately one-sixteenth inch for a 100-foot slab. A similar study of permanent growth of concrete pavement was made at the Arlington Experiment Farm, Virginia, by the Bureau of Public Roads [3] on a 40-foot, plain concrete test section. Over a 9-year period this test section showed a permanent increase in length equal to approximately three-eighths inch for a 100-foot slab. This value is approximately six times greater than the value computed from the data of the reinforced short sections of the Indiana pavement. Whether the reinforcing steel in the Indiana sections restrained the tendency for the concrete to grow in the presence of moisture or whether differences in material and exposure between the sections of the two investigations were responsible for this difference in behavior can only be a matter of speculation.

[3] *The structural design of concrete pavements—part 2,* by L. W. Teller and E. C. Sutherland; Public Roads, vol. 16, No. 9, Nov. 1935; and *Application of the results of research to the structural design of concrete pavements,* by E. F. Kelley; Public Roads, vol. 20, No. 6, Aug. 1939.

Effect of Moisture

Although the data in figure 11 were obtained primarily for a study of length changes of a permanent nature, values of winter-to-summer length changes resulting from the change in the moisture content of the concrete alone can be obtained from them with considerable accuracy. For example, the range in the observed unit length changes of the short sections, as determined from the expansion and contraction periods shown in the figure, is 0.000205 to 0.000240 for the 60° F. change in slab temperature. Inasmuch as the short sections were structurally intact and were relatively free to move, the length changes are principally those caused by changes in the temperature and moisture content of the concrete. It will be recalled that, earlier in the report, the average value of the thermal coefficient of the concrete was estimated to be approximately 0.0000048 per degree F. From this, the unit length change of the sections for a 60° F. change in temperature can be calculated and applied as a correction to the observed unit length change, yielding the unit length change caused by the change in the moisture content of the concrete.

In this investigation the unit length changes caused by the annual cycle of moisture variations were found to range from 0.000048 to 0.000083, these length changes being opposite in sense and partly compensatory for those caused by the annual cycle of temperature changes. For the Indiana pavement, these values correspond to length changes produced by a 10° to 17° F. change in pavement temperature. The values should be approximately a maximum for the yearly cycle, since, as remarked before, the data were obtained at times when the maximum and minimum amounts of moisture were present in the concrete.

Again referring to data obtained from the 40-foot, plain concrete test section of the Arlington experiment, it was found that seasonal variations in the moisture content of that concrete caused length changes corresponing to a 20° to 40° F. change in slab temperature. Hence, it is evident that the effect of moisture on length changes was less for the reinforced sections in Indiana than for the plain concrete section of the Arlington study. In this comparison, also, it seems quite possible that the reinforcing steel restrained, to some extent, the tendency of the concrete to change in length with moisture change.

As a matter of interest, it is noted that in tests conducted by the Minnesota Department of Highways [4] seasonal moisture variations caused length changes corresponding to slab temperatures that averaged 20° F. Also, it was determined in tests by the Michigan State Highway Department [4] that, for a constant temperature of 72° F., the average unit change in length of plain concrete specimens from an oven-dry to a saturated state was 0.000246, this value being equivalent to a change in temperature of 46° F. It appears that different concretes may vary considerably in this characteristic.

The progressive or permanent changes in the length of several of the longer sections are given in figure 12. These data were obtained at the same temperatures and on the same days as those of the short sections discussed previously. The lengths of the individual bars indicate over-all changes in section length that accompanied a 60° F. winter-to-summer rise in pavement temperature. The solid line drawn through the mid-points of the individual bars defines the progressive changes in the lengths of the sections.

It is apparent from this figure that the lengths of all sections increase progressively with time, and that the magnitude of these progressive increases becomes greater with increase in section length for sections up to approximately 1,000 feet long.

The progressive increases of the long sections are the result not only of the tendency of concrete to grow when exposed to cycles of moisture and temperature change, as was the case of the short, uncracked sections; but also of the tendency of transverse cracks to open (however slightly) with time, and of the influence of subgrade resistance. The fact that the long sections returned so nearly to their original or base lengths during the early cycles of length change indicates that the initial widths of the many transverse cracks that

[4] *Investigational Concrete Pavements,* Progress reports of cooperative research projects on joint spacing; Highway Research Board, Research Report No. 3B, 1945.

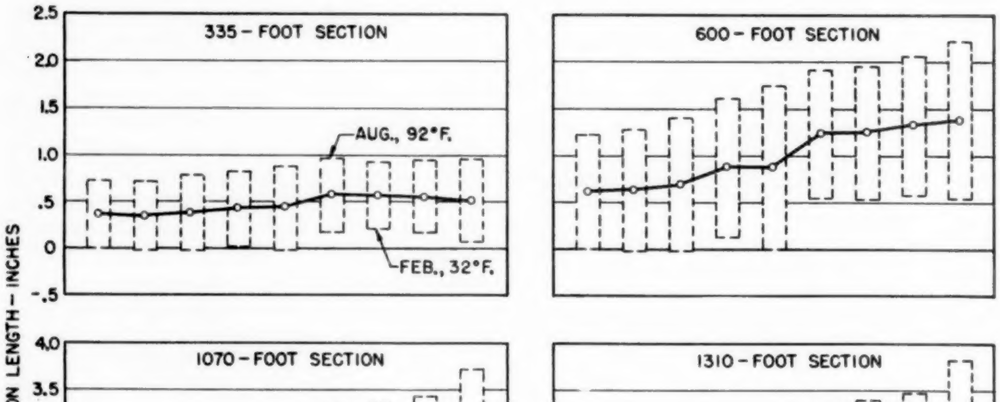

Figure 12.—Annual and progressive length changes of several of the longer sections.

developed during this period must have been extremely small. Also, when the magnitude of the total increase in section length or growth of these sections is divided by the number of cracks in the section, it is evident that the steel reinforcement has prevented any appreciable opening of the individual cracks.

Part 3.—DEVELOPMENT AND DISTRIBUTION OF CRACKS

Five crack surveys were made over the full length of the experimental pavement during the 10-year period. The first was made shortly after the sections were placed; others were made at the end of the first, third, fifth, and tenth years of service. In addition, during the first 3 years of the life of the pavement, certain representative sections were surveyed at more frequent intervals. In every case the surface of the pavement was subjected to a very careful examination in order that all fractures visible to the naked eye might be detected.

Figure 13, traced from the crack survey sheets, shows the number and position of the cracks that have developed in typical sections during the 10-year period of service. Considerable care was exercised in accurately plotting each crack on the original survey sheets. Because of the fine character of the cracks, it was necessary to outline each crack with keel on the surface of the pavement before plotting on the sheets.

It will be noted from the examples in figure 13 that short sections tend to be comparatively free of fractures. At the end of 10 years, 70 percent of 154 short sections—that is, those whose lengths range from 20 to 120 feet—were still uncracked. As the section lengths increase, however, cracking becomes more prevalent until in the central portion of long sections the crack interval is frequently less than 2 feet.

It may be observed also, from the crack patterns shown by the survey sheets, that: (1) Cracks, although somewhat wavy and irregular, are essentially at right angles to the axis of the pavement; (2) cracks in many instances are not continuous across both lanes, either ending completely or being offset slightly at the center joint; (3) longitudinal cracking has not developed in any part of the pavement; and (4) corner breaks at transverse cracks are very rare.

After 10 years of service the surface condition of the pavement is excellent. With the exception of those in sections reinforced with the 32-pound wire fabric, all fractures have been held closed by the longitudinal steel. The cracks that formed in the sections containing this light fabric were wider initially than those that appeared in the more heavily reinforced sections and, after about 8 years, in several cases the steel crossing them broke, probably from shearing forces, resulting in relatively wide openings and some spalling. In all of the other sections there is no evidence of any form of structural damage to the concrete, with the exception of a very slight raveling of the edges of the cracks, probably due to flexure. It is believed that the fineness of the cracks, especially those in the more heavily reinforced sections, is conducive to distributed interfacial pressure, thus minimizing the possibility of blow-ups and other pressure concentration failures that are sometimes observed at cracks in plain concrete pavement.

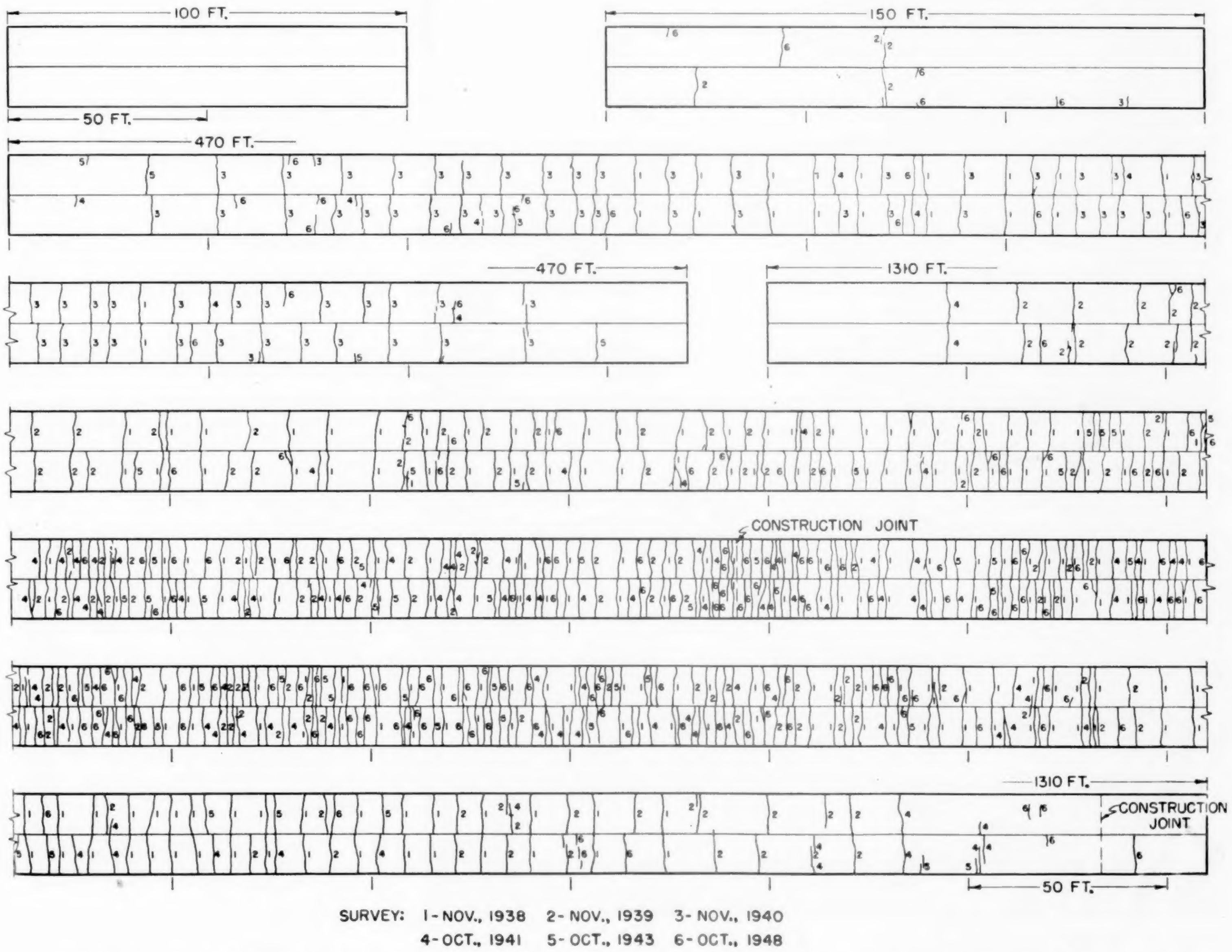

Figure 13.—Typical formation of cracks during the first 10 years of pavement life (sections placed September–October 1938).

Surface Condition at Cracks

Figures 14 and 15 show typical examples of the surface condition of the pavement at several cracks.

Figure 14 pictures the surface condition of the pavement in the vicinity of several of the widest cracks to be found in the central region of the heavily reinforced sections. These cracks are in the right-hand or heavily traveled lane of the pavement and were taken after a 7-year service period. Unfortunately, since the photographs were taken, many of these cracks were inadvertently covered with bituminous material by a maintenance crew. This material did not enter the cracks but did spread over the surface and obscure them.

In figure 15 are shown close-up photographs of the surface condition of the pavement at two cracks; one in the central portion of the longest section of those reinforced with 1-inch diameter rail-steel bars (1.82 percent of longitudinal steel) and the other in a comparable portion of the longest section of those containing ½-inch diameter rail-steel bars (0.45 percent steel). Both photographs were taken after the pavement had been in service for 10 years and show fractures typical of those that appeared in the heavily traveled lane shortly after construction.

The contrast between the surface widths of the two cracks pictured in figure 15 is quite obvious when actually seen in the pavement. When the cracks first formed, those that appeared in the most heavily reinforced sections were almost microscopic, being discernible only by extremely close inspection. However, in the sections with decreasing percentages of reinforcing steel the cracks were, in general, proportionately less frequent and more readily seen; cracks in the lightly reinforced short sections, if present at all, being relatively conspicuous. Over the 10-year period of service the action of traffic and exposure has produced some slight raveling and rounding of the edges of the fractures.

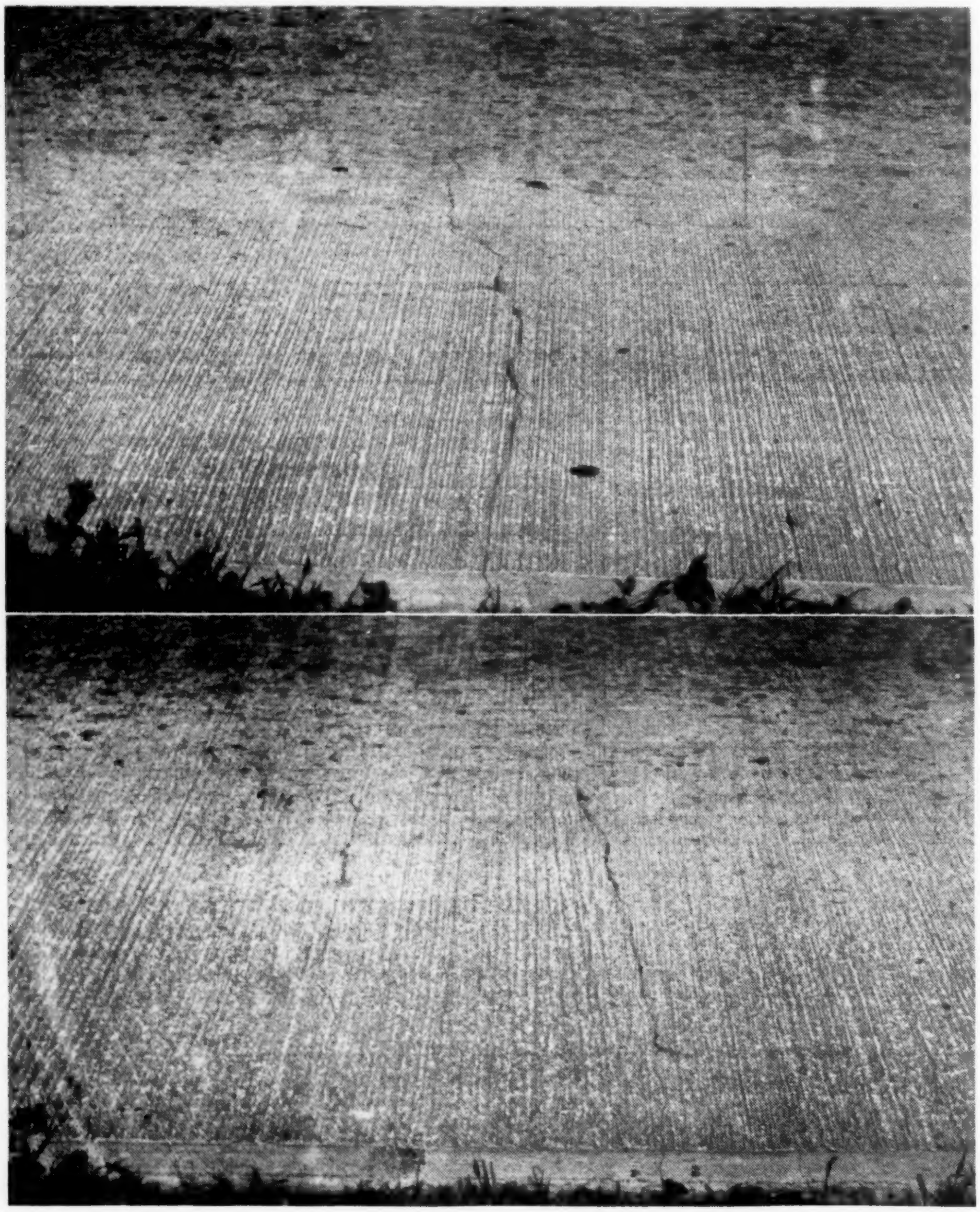

Figure 14.—Surface condition of pavement in vicinity of the widest cracks observed in the central portion of the heavily reinforced sections (heavily traveled lane after 7 years of service).

The preceding discussion related to differences between the surface widths of fractures in sections containing different percentages of longitudinal reinforcement. During the surveys, it was observed further that cracks in the end portion of a given long section generally presented a slightly better surface appearance than those in the central part; and that cracks in the central portion of sections containing a given percentage of steel, but of different lengths, showed some slight evidence of a corresponding difference in surface widths, those in the central part of the longest section of each group apparently being wider than those in the central part of the shortest section. It seems reasonable that this should be so.

Quantitative Measurements

At the end of 10 years, quantitative measurements of the surface widths of cracks, which included raveling and rounding of their edges, were made in the following manner: Starting at the edge of the pavement the width of a segment of crack about 3 feet long was carefully examined and a width measurement made at a point judged to be average. A similar measurement was made on each of two additional 3-foot segments of the same crack, thus covering one lane width. The average of the three measurements was considered to be the average surface width of the crack for the particular lane. All measurements were estimated to the nearest 0.01 inch. It is realized that this procedure does not establish an exact value for the surface width of an individual crack, but it is believed that the averages of a number of such measured values have significance in relative comparisons.

Measured values of the surface widths of cracks obtained in the manner described are given in table 4. The values shown, for each percentage of steel, are of fractures that developed at an early age in the central area of the longest section reinforced with either rail-steel or billet-steel bars. Hence, the computed maximum steel stress was either 45,000 or 55,000 pounds per square inch. An average width value represents the combined average of 15 or 20 cracks, measured in sections containing both rail-steel and billet-steel bars. All data were obtained in the fall of the year when the mean pavement temperature was 58°–60° F.

Table 4.—***The surface width of cracks (central portion of longest section for each percentage of steel)***

Percentage of longitudinal steel in section [1]	Surface width of cracks in lane carrying—			
	Heavy traffic		Light traffic	
	Average	Range	Average	Range
Percent	*Inches*	*Inches*	*Inches*	*Inches*
1.82	0.053	0.02–.11	0.020	0.01–.03
1.02	.078	.03–.15	.032	.02–.05
.45	.104	.07–.18	.038	.02–.06
.26	.117	.09–.15	.038	.02–.07

[1] Calculated maximum stress in steel is either 45,000 or 55,000 pounds per square inch.

The comparisons available in the table show that the surface widths of the cracks tend to increase with a decrease in the amount of longitudinal reinforcement. For example, the average measured width of the cracks in the heavily traveled lane of the selected sections reinforced with 0.45 percent steel is approximately twice the average width of those in sections containing 1.82 percent of steel. The influence of traffic on the surface width of the fractures is also evident, but this effect will be discussed later in the report.

As previously mentioned, the values of table 4 are for sections containing rail- and billet-steel bars. Measurements of the surface widths of cracks also included fractures in the longest section of each group reinforced with the 91- and the 149-pound welded-wire fabric. The data from these measurements are concordant with those from the sections reinforced with rail- and billet-steel bars. The average surface width of the cracks in the section containing the 91-pound fabric was found to be appreciably greater than that of the section reinforced with the 149-pound fabric.

Referring to the range in the average surface width of individual cracks (table 4), it is apparent that the maximum is, in some cases in the heavily traveled lane, slightly more than one-eighth inch. Also, the width of a crack at isolated points along its length was often observed to be considerably greater than its average width, because of localized raveling. The maximum values at such points were 0.3 and 0.7 inch, respectively, for sections reinforced with 1.82 and 0.45 percent of steel. It should be kept in mind, however, that the depth of raveling along the lengths of all cracks was estimated to be never more than one-eighth inch and may be considered superficial.

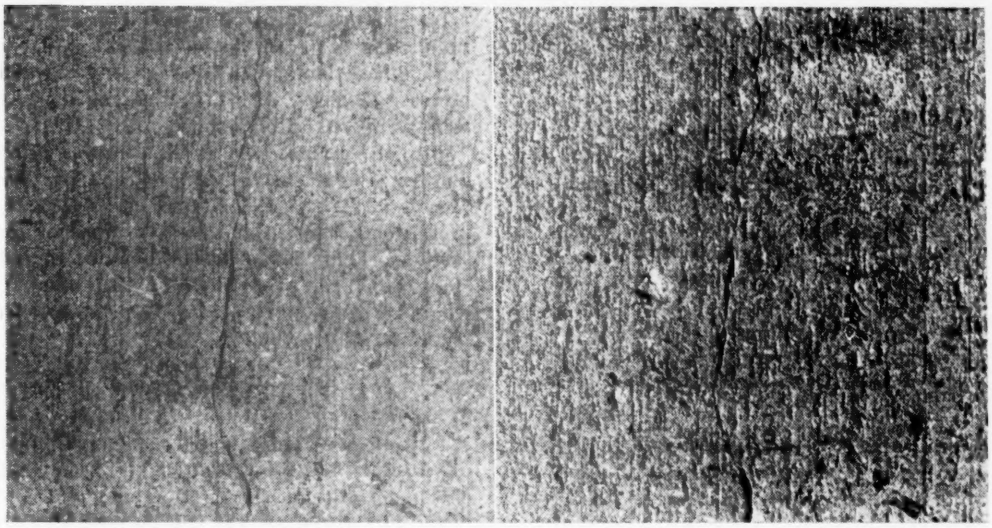

Figure 15.—Surface condition of cracks typical of those that developed at an early age in the central portion of the longest section reinforced with:* (left) *1-inch diameter bars (1.82 percent steel); and* (right) *½-inch diameter bars (0.45 percent steel). These were in the heavily traveled lane after 10 years of service.

A limited amount of supplementary data on the surface widths of fractures, other than those given in table 4, were obtained by measurements in the end and central areas of the 1,310-foot section, in order to establish a comparison of crack widths in those regions. It was found that the surface widths of cracks in the central portion of the 1,310-foot section averaged about twice the width of those near the ends. Measurements were also made of the surface widths of cracks that had developed in the central part of the 600-foot section reinforced with 1.82 percent of steel and such widths averaged about one-half the width of those that formed in the central part of the 1,310-foot section containing the same percentage of reinforcement.

Real Widths of Cracks

Figure 16 shows close-up photographs of several cracks as observed at the vertical face at the edge of the pavement. These photographs, taken in 1948, show fractures that occurred at an early age in the central area of the longest section of each group reinforced with 1-inch diameter rail-steel bars, ½-inch diameter rail-steel bars, and a 91-pound wire fabric, the percentage of reinforcement values being 1.82, 0.45, and 0.24, respectively.

Cracks, such as those pictured in figure 16, were almost imperceptible when they first appeared, being visible throughout the depth of the slab only after a drying period following a wetting of the concrete. With time, however, they have opened progressively a very small amount and their edges have raveled slightly.

At the end of 10 years of service, measurements were made of the edge-face widths of a number of cracks (located in the central portions of the sections mentioned above) in order to obtain values of the real widths of the cracks themselves; width values which, unlike those taken on the surface of the pavement, did not include raveling and rounding of the crack edges. A 40-power shop microscope with a 0.001-inch graduated scale was used to make the measurements, the instrument being focused into the opening of the fracture to eliminate errors caused by surface conditions at the crack edges.

The data obtained from this study of crack widths in the slab edges are given in table 5. Each average value of the table is the average for five cracks that developed early in the life of the pavement. The computed maximum steel stress at the site of these cracks is 55,000 pounds per square inch. All measurements were made at the mid-depth of the slab and in the fall of the year when the mean pavement temperature was 73°–74° F.

The values shown indicate that the real widths of the cracks, like their surface widths, increase with a decrease in the percentage of longitudinal reinforcement. For example, the

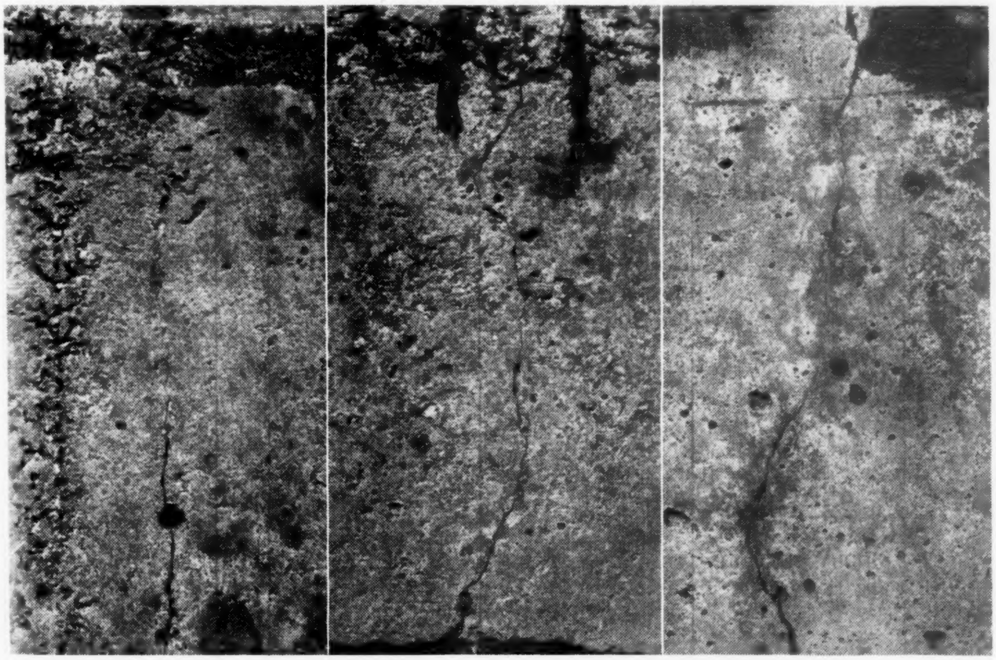

Figure 16.—Edge (vertical face) condition of cracks typical of those that developed at an early age in the central portion of the longest section reinforced with:* (left) *1-inch diameter bars (1.82 percent steel);* (center) *½-inch diameter bars (0.45 percent steel); and* (right) *91-pound wire fabric (0.24 percent steel). These were in the heavily traveled lane after 10 years of service.

Table 5.—*The real width of cracks (central portion of longest section for each percentage of steel)*

Percentage of longitudinal steel in section [1]	Width of cracks in lane carrying—			
	Heavy traffic		Light traffic	
	Average	Range	Average	Range
Percent	*Inches*	*Inches*	*Inches*	*Inches*
1.82	0.004	0.002-.007	0.002	0.001-.003
.45	.011	.007-.018	.009	.007-.010
.24	.013	.005-.018	.010	.006-.013

[1] Calculated maximum stress in steel is 55,000 pounds per square inch.

average width of the fractures in the heavily traveled lane of the selected section reinforced with 0.45 percent of steel is nearly three times the average width of those in the same lane of the section containing 1.82 percent of steel. A comparison of the data in tables 4 and 5 shows that the surface width of cracks increases under the same conditions that cause an increase in real width. It is apparent, also, that the surface width of a given crack is many times greater than its real width.

Longitudinal reinforcement is in continuous bond with the concrete until the first transverse crack develops. When this happens the amount of opening of the crack will depend upon the total elongation of the steel which crosses it. This elongation is, in turn, dependent upon the length that is free to elongate as affected by the bond between the steel and the concrete; and upon the magnitude of the direct tensile stress in the steel, also dependent upon bond conditions.

In this investigation neither the length over which the steel was not in bond nor the magnitude of the tensile stress in the steel could be determined.

However, it is of interest to examine the crack-width data on the basis of the amount of longitudinal steel present, as shown in table 5. Presumably, at the time of the crack-width measurements the same steel stress was active in the central region of all sections listed in the table. When compared in this way it will be found that for both the heavily traveled and the passing lanes, the average crack width increases directly with a decrease in the percentage of longitudinal steel.

Also of interest is the fact that in the longer sections the surface widths of cracks, and presumably their real widths also, were less in the end than in the central areas of the sections. This is as would be anticipated, since the tensile stress in the longitudinal bars would be expected to decrease as the end of a section is approached.

Effect of Traffic

In connection with the study of cracking, an opportunity has been afforded to observe the effect of traffic on the development and condition of the cracks. It will be recalled that the experimental two-lane pavement is one-half of a divided highway; consequently, the right-hand lane carries the greater number of vehicles and practically all of the heavy trucks, the left-hand lane being used largely for passing. Also, it is mentioned again that the experimental sections, part of U S 40, are subjected to a relatively high frequency of heavy traffic loads.

Although a survey made soon after completion of the pavement showed equal cracking in both lanes, at the end of the first year 51.2 percent of the total number of cracks were found to be in the right-hand lane of the pavement. This percentage value had increased to 52.7 and 53.0 percent at the end of the fifth and tenth years, respectively. Thus, it appears that repetition of traffic loads has exerted a slight but only a slight influence on the development of transverse cracks. Since approximately two-thirds of the total or present number of cracks formed during the first year, when only 51.2 percent formed in the right-hand lane, the effect of traffic repetition on subsequent cracking is somewhat more pronounced than is indicated by the preceding percentage values.

Figure 17 was prepared to show the influence of traffic on cracking during specific periods of the life of the pavement. For the periods indicated by the circled numbers, each individual bar represents the number of cracks that formed in the heavily traveled right-hand lane expressed as a percentage of those that formed in both lanes of the pavement. It will be noted that an equal number of cracks appeared in both lanes of the pavement within the first month or two after construction. During each subsequent period a progressively greater number of cracks formed in the heavily traveled lane than in the passing lane until a maximum value of 62 percent was reached for the period covering the third to fifth years of pavement life. During the last 5 years about 55 percent of all new cracks developed in the right-hand lane of the pavement.

Also, as will be seen by an examination of the data in tables 4 and 5, traffic has had an

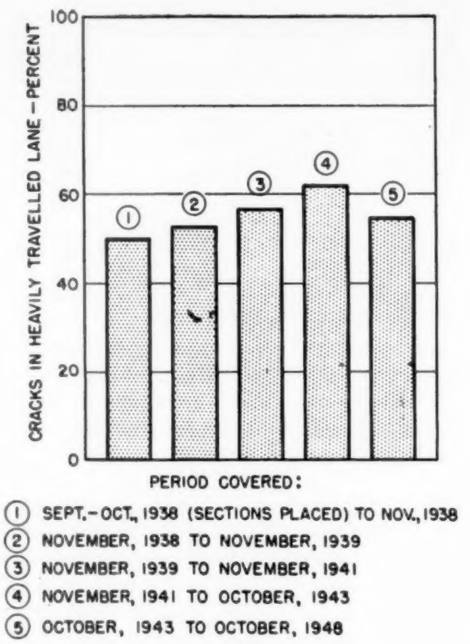

Figure 17.—Effect of traffic on the amount of cracking.

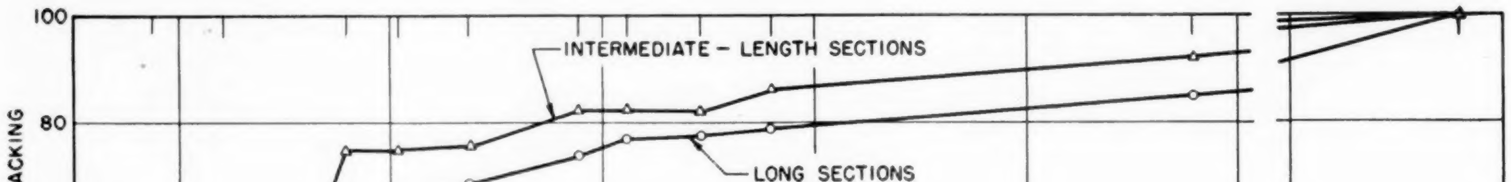

Figure 18.—Rate of crack development during the first 10 years of pavement life.

appreciable effect on both the surface and the real widths of the transverse cracks. Comparisons of the average widths of cracks in the right-hand lane with those of companion cracks in the passing lane are given in table 6.

It is apparent that the heavier traffic using the right-hand lane has produced more extensive raveling and other superficial damage at the crack edges and a wider separation of the fractured faces than the lighter traffic on the left-hand lane. This effect of traffic is naturally more pronounced in the case of the surface widths of the cracks.

Rate and Distribution of Cracking

Figure 18 shows the manner in which cracking has developed with respect to periods of time. In this figure the sections were grouped according to length, as short, 20–120 feet; intermediate-length, 120–470 feet; and long, 470–1,310 feet.

Thirty-one percent of the total or present number of cracks in the long sections and 11 percent of those in the intermediate-length sections appeared within approximately 1 month after construction. Few cracks occurred during the first winter, none in the short sections. However, the rate of cracking was quite high for all groups during the period that followed, a period that included the interval between late March and late October of the first year.

The survey made at the end of this first year of service showed 65 percent of the present cracking had developed in the long sections, 75 percent in the intermediate-length sections, and 25 percent in the short sections.

On the basis of all transverse cracks that have developed during the 10-year period of service, it is of interest that 67 percent had appeared by the end of the first year. After the first year the rate has been quite low and, in general, rather uniform. Between length groups the highest rate has been in the short sections.

In figure 19 is shown the distribution of cracking for representative sections, expressed as the number of cracks per 50 feet of section. The data indicate that the number of cracks per 50-foot increment increases from a minimum value at the end of a section to a maximum value in the central area, in a generally normal frequency distribution pattern; and that the maximum values, as found in the central area of the sections, increase progressively with increase in section length.

It will be noted that the symmetry of cracking in the experimental sections is not only indicative of structural uniformity, but also implies that the nonuniformity of the elevation changes that developed in the pavement, as mentioned earlier in the report, apparently had little effect on the formation of cracks.

The manner in which cracking developed in the sections is, in some respects, shown to better advantage in figure 19 than in figure 18, especially since the distribution of cracking for the various time periods is given in the latter figure.

The magnitude and distribution of the cracking that appeared within approximately 1 month after construction of the sections are shown as the first time period. During this period no cracks were found in sections having lengths of 210 feet or less, and only a limited number in the central portion of sections with lengths between 270 and 360 feet; but a considerable number were found in the 600- and 1,070-foot sections at some distance from the ends. Since the cracking during this period appeared only in the central areas of the longer sections, it is believed that it had its origin primarily in the tensile stresses induced by subgrade resistance during shrinkage of the sections either from loss of moisture, decrease in pavement temperature, or both.

The second time period covers the first winter after construction. The survey at the end of the winter indicated that sections having lengths of 210 feet or less were still uncracked and that only a small amount of cracking, spottily distributed, had developed

Table 6.—*Comparison of average widths of cracks in right-hand lane with widths of companion cracks in left lane*

Percentage of longitudinal steel in section	Ratio of crack width in right-hand lane to that in left lane.
SURFACE CRACK WIDTH	
0.26	3.1:1
.45	2.7:1
1.02	2.4:1
1.82	2.7:1
REAL CRACK WIDTH	
0.24	1.3:1
.45	1.2:1
1.82	2.0:1

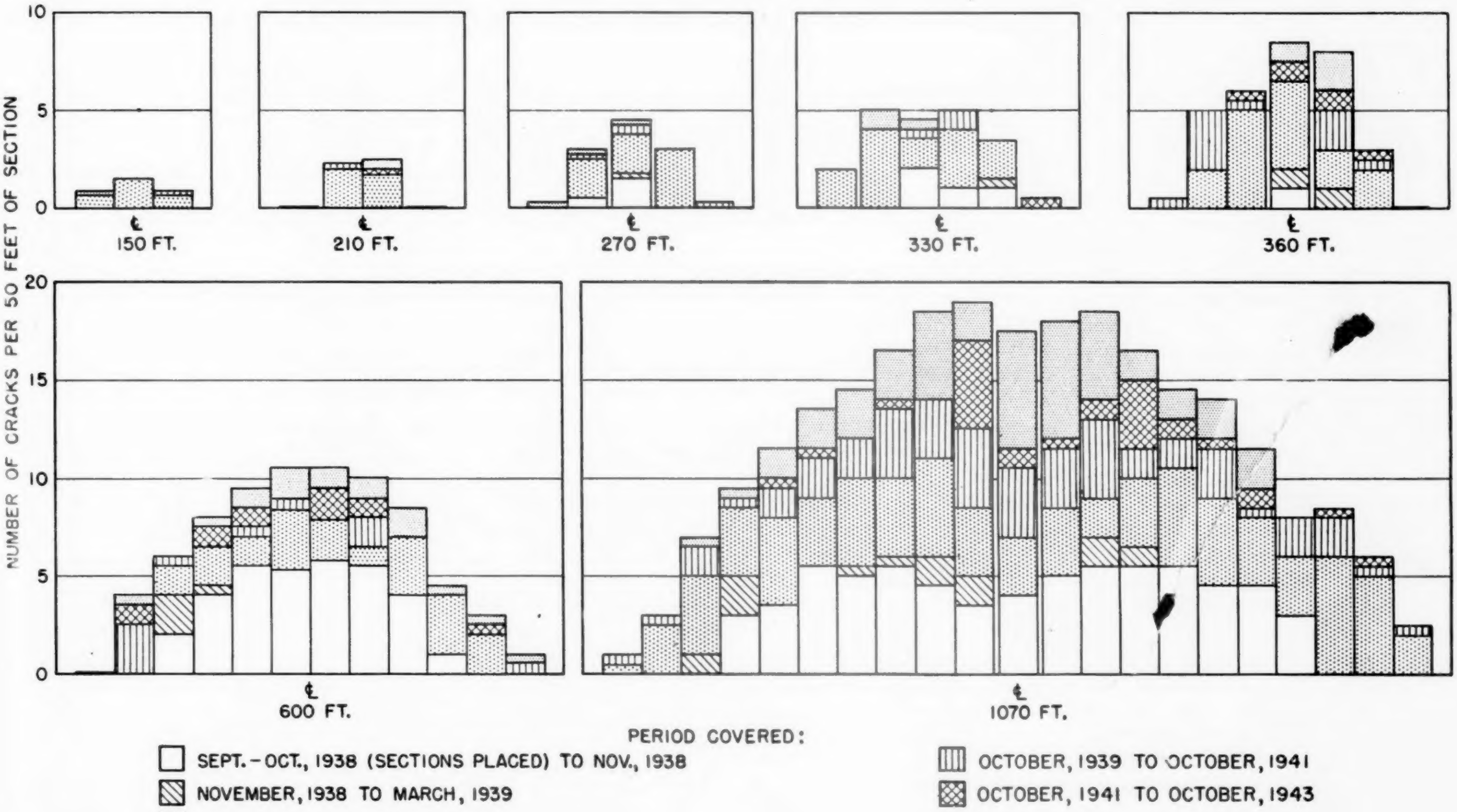

Figure 19.—Distribution of cumulative cracking per 50 feet of section length in the first 10 years of pavement life.

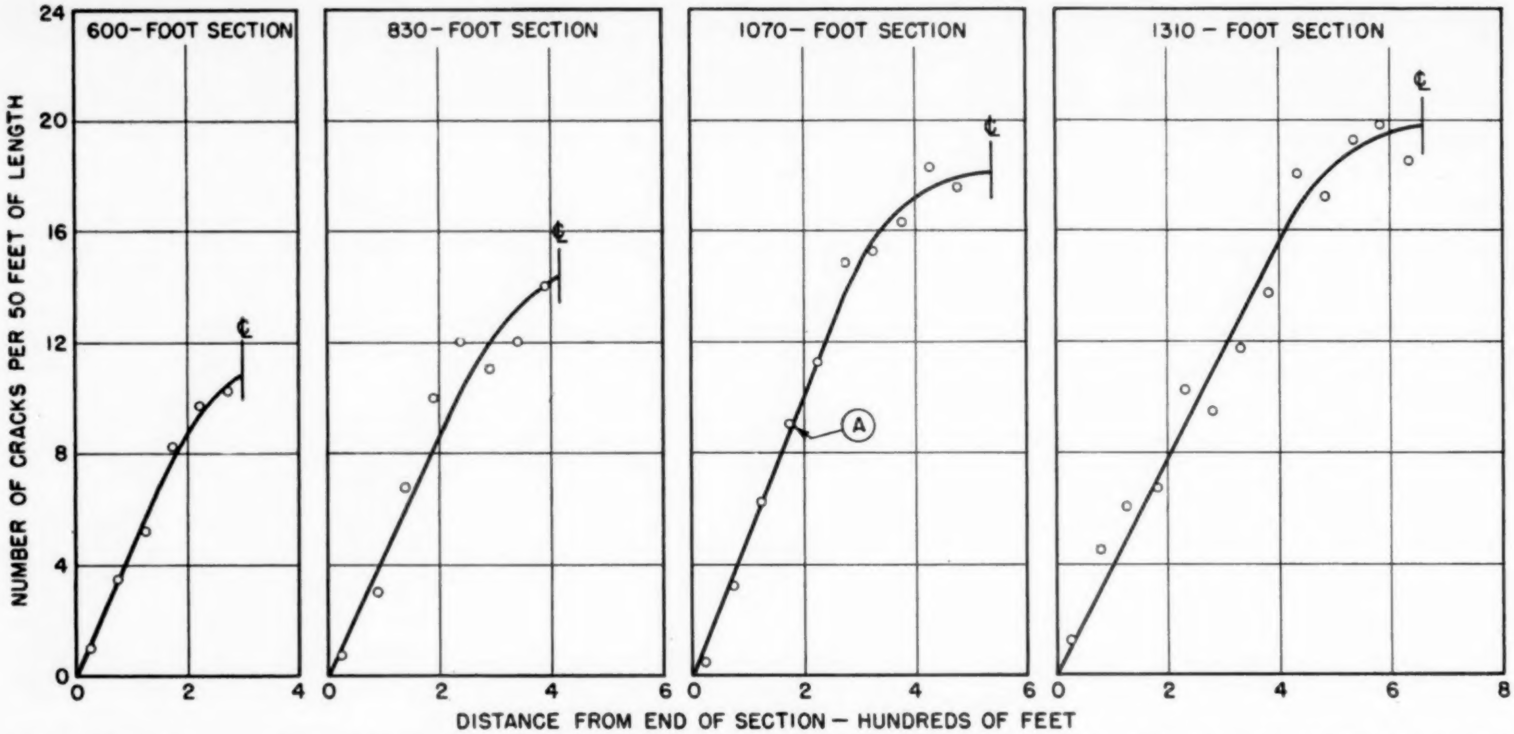

Figure 20.—Frequency distribution of cracking at the end of 10 years for sections reinforced with 1-inch diameter rail-steel bars (1.82 percent steel): average of both lanes.

in sections having lengths equal to or greater than 270 feet. The relative absence of crack development during this period indicates that the nonuniform changes in pavement elevation caused by frost penetration of the subgrade had little influence upon cracking; and that tensile stresses from subgrade resistance were no greater during the winter period than during the preceding fall. This supports the conclusion drawn from the data of figure 9.

In the third time period, between late March and late October of the first year, a noticeable change occurred in the crack development in all of the sections. In fact, a large percentage of the cracks now present in sections having lengths of 360 feet or less, and in the end areas of the longer sections, formed sometime during this period. Such cracking is believed to have been caused primarily by stresses induced by restrained warping. Unfortunately, the pavement was not surveyed in midsummer so it is not possible to determine more closely the part of this time period during which cracks formed. However, it is suspected that the cracks developed in large part during late spring and early summer when warping stresses are generally highest for the year. The fractures that formed at some distance from the ends of the longer sections may have resulted, also, from stress combinations existent in early fall when the sections were contracting after having attained their maximum annual unrestrained lengths.

During the fourth time period, which covers the second and third years after construction, the development of new cracks was confined primarily to the central areas of the long sections. The relatively small number of fractures that appeared during this period and during the succeeding fifth time period (fourth and fifth years) greatly reduced the rate of crack development. Within the sixth and last time period of this study, from the fifth through the tenth years of pavement service, the greatest number of cracks again have formed in the central areas of the long sections, suggesting a continued high stress condition in those regions.

Crack Frequency Patterns

Frequency distribution curves for the cracking that existed at the end of 10 years in the four sections comprising the group reinforced with 1.82 percent of longitudinal steel (1-inch rail-steel bars) are shown in figure 20.

The ordinate values represent the number of cracks per 50 feet of section and the corresponding abscissas are distances from the end of the section to the centers of the 50-foot lengths to which the ordinate values apply. For example, at point A in the figure there are nine cracks in the 50-foot length which lies between 150 and 200 feet from the end of the 1,070-foot section.

It is apparent that: (1) For some distance, beginning at the end of a section, the crack frequency for successive 50-foot increments increases directly with increase in distance; (2) the length over which the linear relation holds increases progressively with increase in section length; and (3) the slopes of the linear portions of the curves appear to be nearly the same for the different section lengths. It is believed that the frequency of cracking in the sections reflects, to a considerable extent, the stress distribution in the longitudinal steel as induced by subgrade resistance.

Frequency distribution curves were constructed for all sections having lengths equal to or greater than 150 feet. From these curves the maximum cracking frequency value (number of cracks per 50 feet of section in the central area) was determined for each section. Figure 21 shows, for each of the three types of reinforcement, the relation established by plotting such frequency values against the corresponding section lengths. In order to show possible effects of the stresses in the steel, the maximum computed steel stresses are indicated by symbol.

It is apparent that the maximum cracking frequency increases with an increase in section length, the relation being nearly linear for section lengths between 400 and 1,000 feet. For section lengths greater than about 1,000 feet the curves depart from linearity, indicating that a condition of complete restraint is being approached. This suggests that sections having lengths somewhat greater than 1,000 feet, possibly the 1,700–1,800-foot length mentioned in the discussion of figure 9, would develop complete restraint in the central region and that sections of this length or greater would have equal maximum cracking frequencies irrespective of their over-all lengths.

The 10-year data indicate that an interval between cracks in this region of complete restraint might be expected to be approximately 2.0 to 2.5 feet.

Effect of Reinforcement Type

The data shown in figure 21 indicate that the type of reinforcement has only a slight effect on maximum cracking frequency. For example, within the length range of sections containing welded-wire fabric, the maximum cracking frequency values are only slightly less than those for sections of comparable length reinforced with billet- or rail-steel bars. Comparing the bar-reinforced sections, it appears that the maximum cracking frequency values are slightly greater for billet- than for

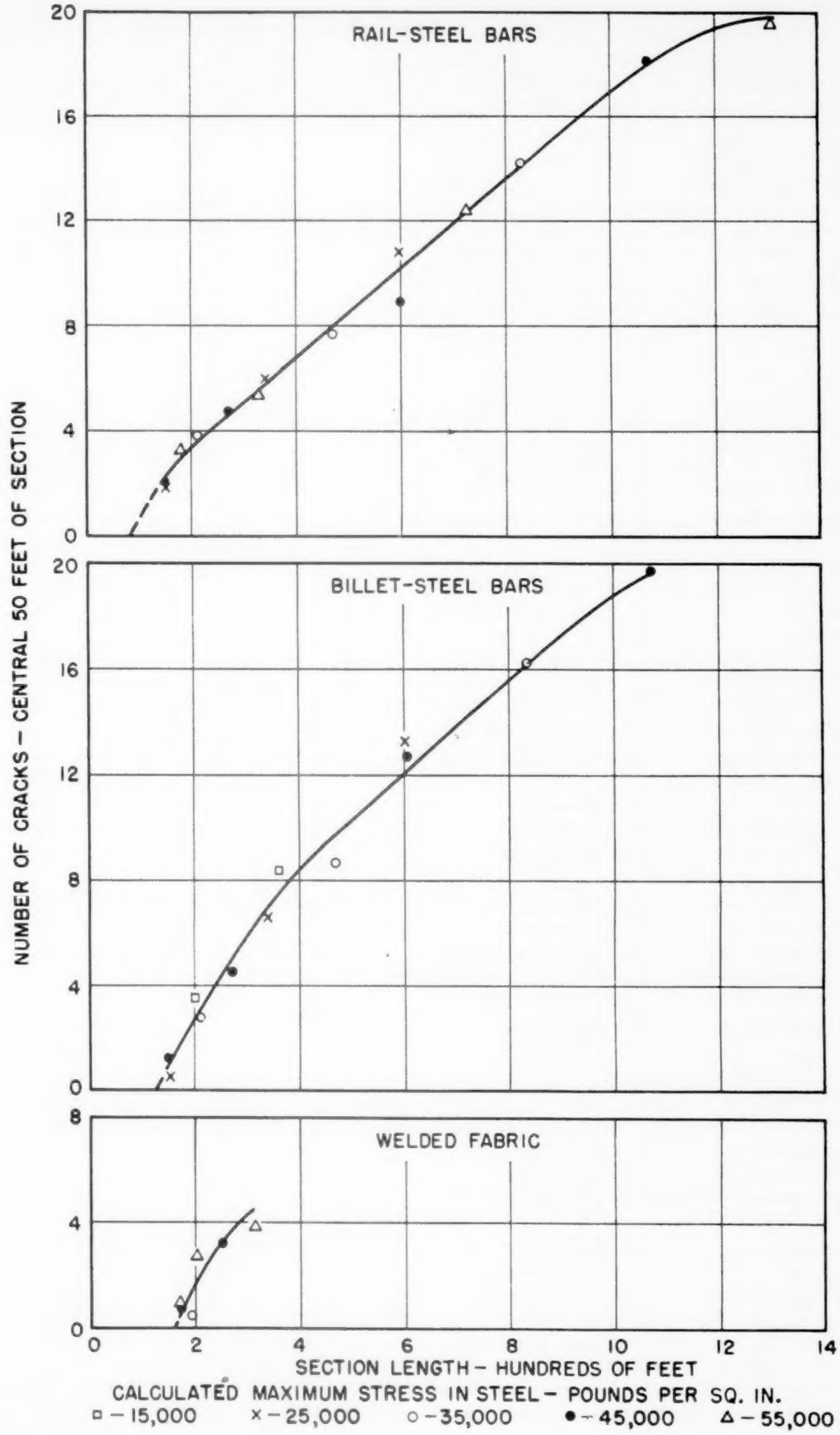

Figure 21.—Effect of type of reinforcement and of calculated maximum steel stress on the relation between section length and maximum cracking frequency at age of 10 years.

rail-steel bars (at 1,000 feet these values are 18.8 and 17.0, respectively).

On the other hand, it will be noted that all symbols denoting the various magnitudes of computed steel stresses fall very close to the mean curves, indicating that, within the range of steel percentages in sections of common length, a variation in the amount of steel is not accompanied by a corresponding variation in maximum cracking frequency.

Because of conservative design assumptions, the relation between amounts of reinforcing steel and the section lengths are such that the steel has, in all probability, never been stressed beyond its elastic limit. Therefore, if reinforcement which is adequate for a given section length, say 400 feet, had been used for the entire range of section lengths, the relations shown in figure 21 would not obtain and differences due to variations in the maximum steel stresses might have appeared. The longer sections would probably either subdivide due to breakage of the steel or contain fewer but wider cracks as a result of inelastic deformation of the steel.

Another analysis of the data that is of interest is shown in figure 22 in which the average slab length, after 10 years of service, is plotted against section length as constructed. In this figure separate curves are given for each of the three types of reinforcement, and the maximum steel stresses as computed during the designing of the sections are indicated by symbol. Slab length is defined as the distance between transverse cracks or joints, all joints being considered as cracks. Each point defining the curves is an average value of either two, four, or six sections.

Although the points defining the curves appear to be somewhat erratic for sections up to approximately 200 feet in length, it is believed that this is a statistical effect caused by the relatively small number of cracks in sections of these lesser lengths.

It is apparent that the three curves of figure 22 follow the same general pattern; that is, the average slab length increases with an increase in section length until a peak value is reached, beyond which there is a rapid decrease in average slab length that becomes more gradual and finally approaches a constant value for the longer sections. In the case of the 1,310-foot section the present value of the average slab length is 4.2 feet. At the end of the first and fifth years this value was 7.0 and 5.1 feet, respectively.

The type of reinforcement has quite an obvious effect on the relation between section length and average slab length, especially in the case of the shorter sections. The greatest average slab length for the sections containing welded fabric is 109 feet, which was reached at an optimum section length of 135 feet. In the case of the billet-steel bars the value is 97 feet, attained at an optimum section length of 115 feet; but for the rail-steel bars this value is only 48 feet, the corresponding section length being 90 feet.

The differences in the peak slab-length values and corresponding optimum section lengths for the three types of reinforcement cannot be fully explained. There are, however, two conditions that may have had some influence on the data shown. First, in that part of the curves which pertains to the shorter sections, there are fewer points defining the curve in the case of the rail-steel bars than in the case of the other types of reinforcement. Second, in all sections reinforced with welded fabric and in all sections, except one, with lengths of 270 feet or less reinforced with billet-steel bars, the coarse aggregate used in the concrete consisted of a mixture of small-size gravel with large-size crushed limestone; whereas in sections reinforced with rail-steel bars the coarse aggregate used in the concrete was entirely the crushed limestone. There is no other evidence, however, that the difference in coarse aggregate mentioned affected in any way the behavior or present condition of the sections.

The data of figure 22 mean that, under the

conditions obtaining in this experimental pavement, the longest average slab lengths are found at the so-called "optimum" section lengths. Hence, if one were interested only in a minimum number of transverse cracks and joints, these data suggest that in reinforced concrete pavements the transverse joints be spaced approximately 100 feet apart. However, as this investigation strikingly shows, a longer section with many transverse cracks can continue to be a strong, durable structural unit after many years of heavy traffic service if it contains an adequate amount of longitudinal reinforcement.

In the relations shown in figure 22, the maximum computed steel stress values apparently have no influence on the amount of cracking in a given section length. This is concordant with the relations shown in figure 21.

Part 4.—BEHAVIOR OF THE SPECIAL SECTIONS

The four special 500-foot sections containing weakened-plane warping joints at 10-foot intervals have been subjected to the same close study as have the regular sections.

It will be recalled that in each of the four special sections, relatively light welded-fabric reinforcement was placed continuously through all of the weakened-plane warping joints over the 500-foot section length. The bond between the steel and the concrete was destroyed purposely for a distance of 18 inches on each side of each joint by omitting two transverse wires, one on either side of the joint, and by greasing the longitudinal wires over the 36-inch length. In addition to the continuous reinforcement, shear bars consisting of ¾-inch diameter dowels 18 inches long, spaced 12 inches center to center, were placed across the warping joints in one-half of each of the four sections.

The distinguishing features of the four 500-foot special sections are as follows:

Section 1.—Weakened-plane joints are of the submerged type and the reinforcement weighs 91 pounds per 100 square feet.

Section 2.—Same as section 1, except that the reinforcement weighs 45 pounds per 100 square feet.

Section 3.—Weakened-plane joints are of the surface-groove type and the reinforcement weighs 91 pounds per 100 square feet.

Section 4.—Same as section 3, except that the reinforcement weighs 45 pounds per 100 square feet.

Through the design features of these special sections it was proposed to develop information on the practicability of a pavement design in which transverse crack control was obtained by means of relatively short slab units (10 feet) with pavement continuity obtained by the use of continuous longitudinal reinforcement. Other information sought pertained to the amount of longitudinal steel necessary to resist the tensile forces created by subgrade resistance in a section of this length; the value of the design feature in which bond was deliberately destroyed for 18 inches on either side of the joint; and the necessity for protection of the longitudinal

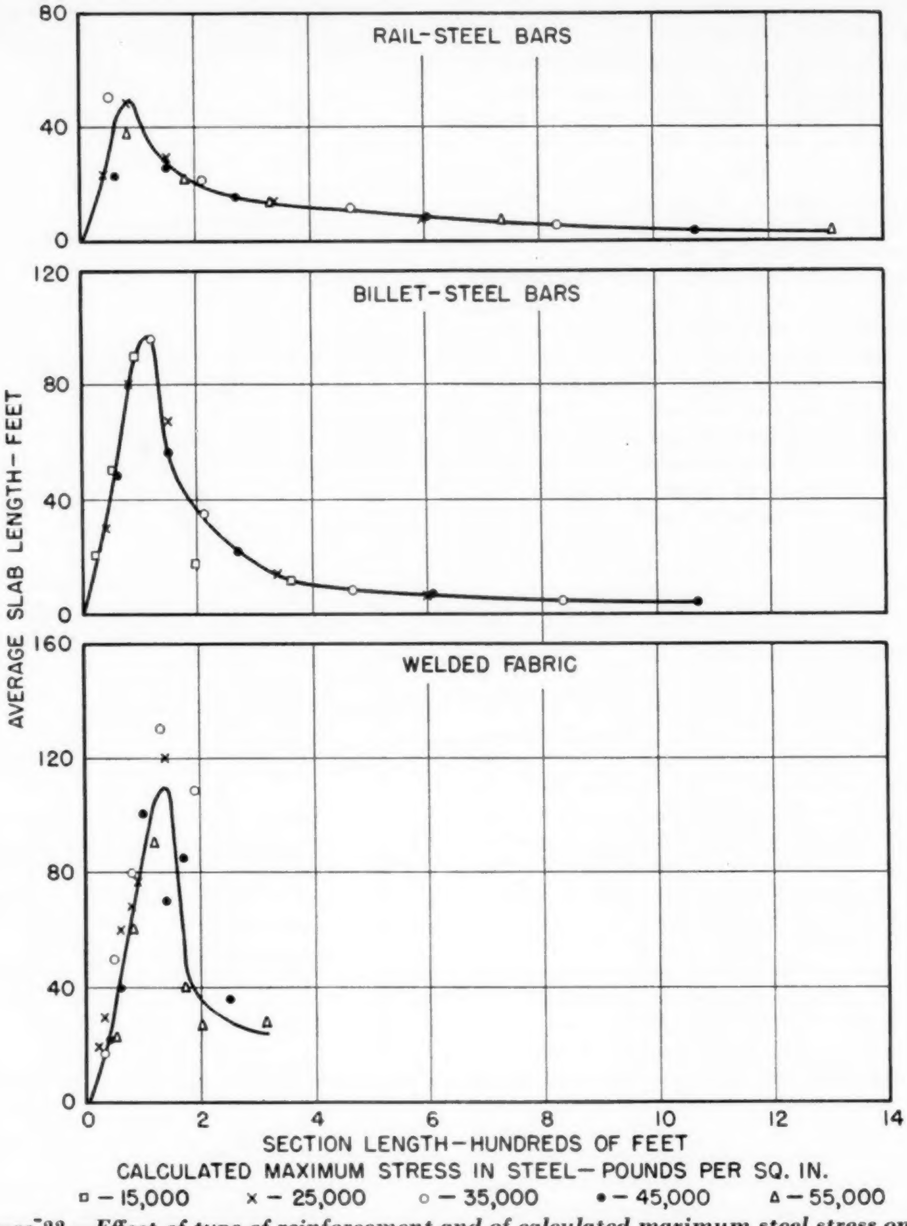

Figure 22.—Effect of type of reinforcement and of calculated maximum steel stress on the relation between section length and average slab length at age of 10 years.

reinforcement against shear in the transverse joints by means of dowels used to develop shear resistance. It was thought that such protection probably would be necessary because of the relatively large joint opening expected from elastic elongation of the longitudinal steel over the 36 inches of unbonded length at the transverse joints.

During a drop in pavement temperature, a continuously reinforced section naturally attempts to contract about the center of the section length. At the same time, the individual segments or slab units of the section are attempting to contract about their individual centers. The amount that these individual segments contract should equal the elongation of the steel crossing the fractures that define their lengths. This elongation of the steel is dependent upon the magnitude of the stress induced in it by resistance as the segments tend to move over the subgrade; and upon the length over which the bond between the steel and concrete is destroyed—that is, the length over which this stress is effective.

Thus, by subdividing the 500-foot special sections into 10-foot slab lengths so that during a large temperature drop the contractive length change of an individual slab unit would be relatively small, and by breaking the bond for 36 inches at each separation between slabs so that the elongation of the steel could be relatively great without exceeding the elastic limit, it seemed that a certain degree of control over the movement of a section should be gained without rupturing the reinforcing steel. For example, during a sudden drop in pavement temperature when subgrade resistance is relatively great, the continuous reinforcement would simulate a steel spring at each transverse joint—elongating and permitting the slab units to contract about their individual centers, and subsequently contracting and drawing the units together as the subgrade resistance decreased.

Failures at Joints

From the standpoint of design, all of the special sections behaved satisfactorily during

Table 7.—*Steel failures at joints in the special sections at the end of 10 years*

Joint type	45-pound fabric	91-pound fabric
Surface type:		
With dowels	0	0
Without dowels	9	2
Submerged type:		
With dowels	2	0
Without dowels	4	1

the first 3 years of pavement service. Then the reinforcing steel began to fail at the joints. During the condition survey at the end of 3 years two breaks in the reinforcing steel were discovered, both at joints in the sections containing the 45-pound wire fabric and both at joints without shear bars, one of the breaks being only 60 feet from the end of a section. After 5½ years the reinforcement was either found to be broken or suspected of being broken at seven of the joints. All of these failures developed at joints without shear bars and in the sections reinforced with the 45-pound wire fabric. At the end of 10 years the reinforcing steel was either broken or elongated beyond its elastic limit at 18 of the joints. The distribution of these steel failures is given in table 7.

From table 7 it will be noted that 16 failures were at joints without shear bars; 15 failures developed in sections reinforced with the 45-pound wire fabric; 7 and 11, respectively, were found in sections constructed with the submerged and the surface-groove type of joints; and none occurred in the halves of the sections provided with shear bars at the joints and containing the 91-pound wire fabric. The effect of such failures will be discussed in parts 5 and 6 of this report.

The fact that all of the earlier failures of the reinforcing steel and approximately 90 percent of those now present occurred at joints having no shear bars indicates that shearing forces caused by loads passing over the joints were primarily responsible for the steel failure. However, it is possible for a progressive separation to develop at a joint and eventually overstress the reinforcing steel. Infiltration of solid material would, in time, cause a permanent opening of the joint and dissipate all or part of the elastic elongation of the steel in the 36 inches of unbonded length across the joint. Subsequently, during contraction periods, the steel, if small in amount as in the 500-foot special sections, might be subjected to direct tensile stresses sufficiently large to cause failures. Such action might account for the two cases of steel failure observed at the joints provided with shear bars. These developed after the pavement had been in service for 8 years. Both occurred at some distance from the ends of sections containing the 45-pound wire fabric and both were at joints of the surface-groove type.

Surface Condition of Weakened-Plane Joints

Figure 23 shows photographs, taken after 10 years, of two of the submerged type, weakened-plane joints at which the reinforcing steel was

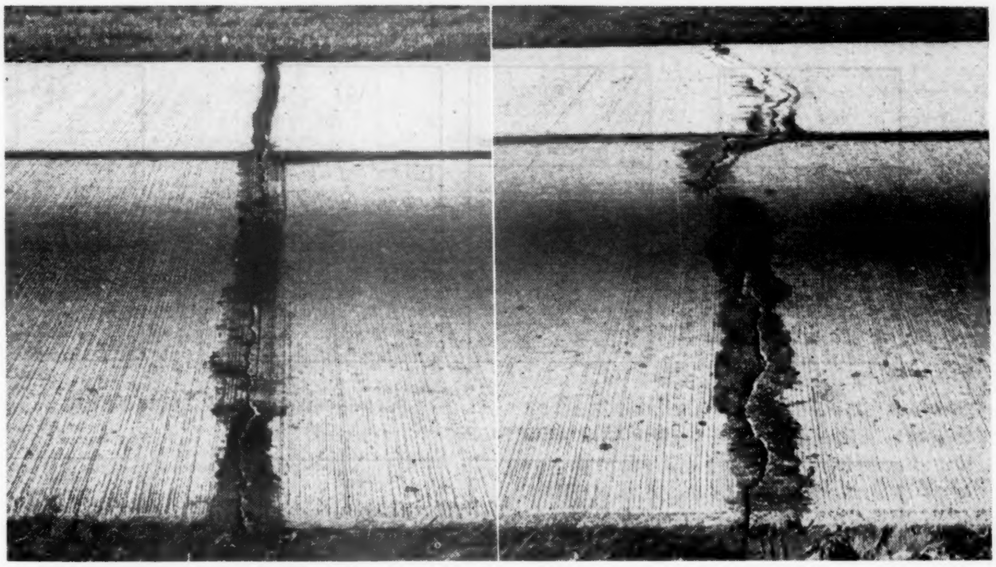

Figure 23.—Two of the submerged-type, weakened-plane joints after 10 years of service: Extreme cases of straight and irregular cracking over the bottom parting strip.

unduly inelastically elongated but not broken insofar as could be determined. These two joints were selected as extreme cases of straight and irregular cracking over the submerged parting strips used in creating this type of joint. The fractures that formed over these strips were, in general, meandering in character and were much wider than those that developed in the regular sections containing comparable percentages of continuously bonded reinforcement. Under the influence of traffic and exposure the edges of the fractures have raveled and spalled to a considerable width, creating a rather unsightly surface condition. This condition developed most rapidly during the first 2 or 3 years of service. Subsequently the deterioration in surface condition has been gradual.

The condition of the weakened-plane joints of the surface-groove type was excellent,

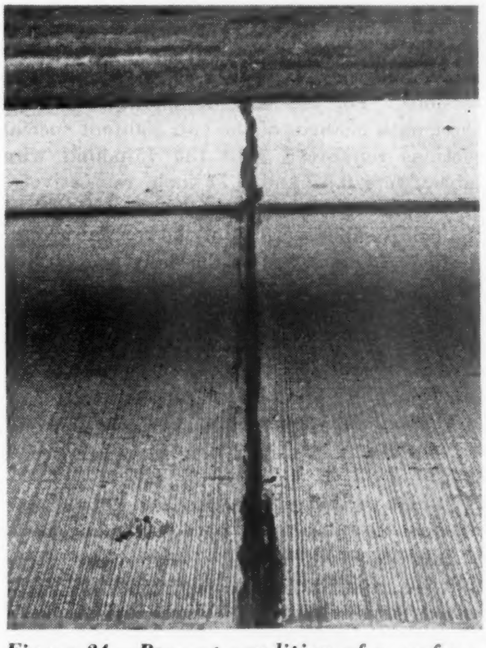

Figure 24.—Present condition of a surface-type, weakened-plane joint, typical of those at which the continuous reinforcement is structurally sound.

initially, and continued to remain so except where there has been failure of the reinforcing steel. The present appearance of a typical joint is shown in figure 24. Very little maintenance has been required at these joints, so long as the steel remained structurally sound, since the comparatively small length changes of the 10-foot units are conducive to well-sealed conditions.

Progressive Changes at Joints

Daily, annual, and progressive changes in the widths of the joints of the four special sections were measured during the same periods as those of the regular sections. In figure 25 are shown the annual and progressive changes in the widths of the joints, plotted with respect to base measurements taken during the first winter after construction. The width changes of the expansion joints are, in reality, length changes of the sections as determined by measurement to fixed reference points located at their ends. Attention is called to the differences in the vertical scales used for the width changes of the expansion and weakened-plane warping joints, this being necessary because of the relatively small magnitude of the width changes of the latter. The lengths of the stippled bars indicate changes in width that occurred at the joints during an annual cycle. These changes should be nearly maximum for such a cycle since an attempt was made to obtain the measurements during the hottest and coldest periods of the year. All measurements in a given section were discontinued as soon as the first failure in the reinforcing steel was noted.

It is apparent in figure 25 that, in spite of the continuity of the reinforcing steel throughout the length of a section, the expansion joints closed and the weakened-plane joints opened progressively with time. These progressive changes are in the same sense as those observed in plain concrete pavements built with expansion joints and closely spaced weakened-plane contraction joints. It has been observed, also, that the progressive clos-

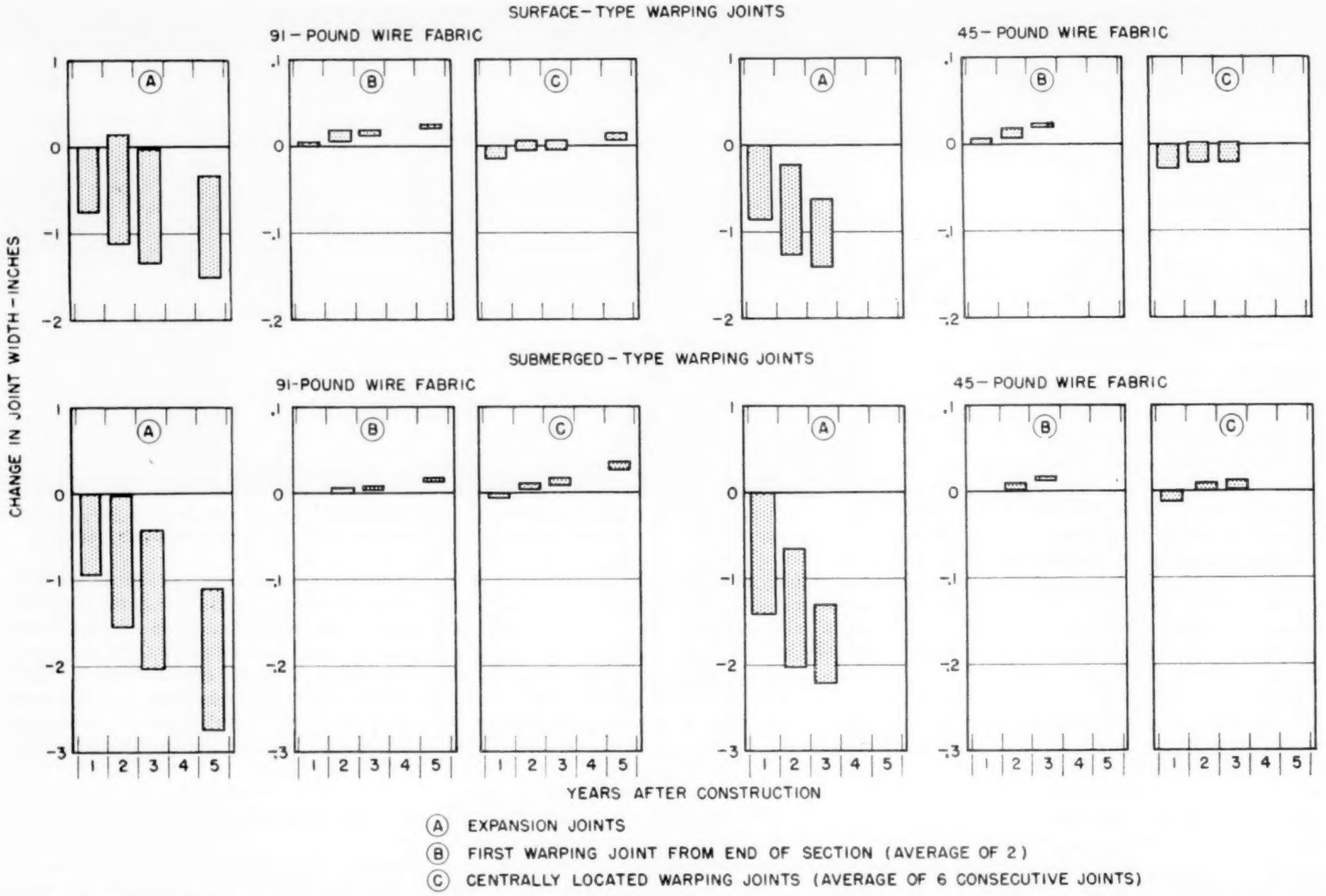

Figure 25.—Annual and progressive joint width changes of the 500-foot sections containing warping joints at 10-foot intervals: Values above the zero line denote opening; those below, closing, of the joint with respect to the base readings of December 1938.

ure of expansion joints at the end of 3 years has been less in sections reinforced with the 91-pound wire fabric than in those containing the 45-pound wire fabric; and less in sections with the surface-type joints than in those with the submerged type. This latter observation implies that extraneous material infiltrates more readily into the submerged-type joints, indicating that joints of this type are more difficult to seal. The behavior of the weakened-plane warping joints is somewhat erratic and, other than the fact that a progressive opening has developed in all cases, clear-cut trends are not apparent.

As remarked before, one of the purposes of the longitudinal steel in these 500-foot special sections was to hold the slab units of the sections together, as much as possible, during contraction periods. In this respect it is clearly shown in figure 25 that the reinforcing steel, especially the heavier fabric, exercises considerable control over the behavior of the sections during such periods. Measurements of the over-all section-length changes at the ends of the two sections reinforced with the 91-pound wire fabric showed 1.09 and 1.13 inches, respectively, for a mean pavement temperature drop of 77° F. that occurred between midsummer and midwinter of the second annual contraction period. This is shown in figure 25 as the difference between the lower end of the second bar and the upper end of the third bar in each of the four graphs marked *A*. These values represent approximately 77 percent of the annual contractive change of a section of equal length, but containing heavier, continuously bonded reinforcement such as was used in the regular sections. For the same temperature change the length changes of the two 500-foot special sections reinforced with the 45-pound wire fabric were 0.67 and 0.71 inch, respectively, or about 48 percent of the length changes of the comparable sections of the regular group. A similar comparison of the daily contractive length changes of the special sections with those of the section containing the continuously bonded steel results in values of approximately 54 and 37 percent, respectively, for the 91- and 45-pound wire fabric.

These comparisons show that the pattern of movement of the ends of the 500-foot special sections, although of less amplitude, is similar to that observed in the regular sections containing continuously bonded steel. It is apparent that during periods of contraction the heavier of the two weights of reinforcement in the special sections was more effective in holding together the individual slab units; and that both weights of reinforcement were more effective during annual periods than during daily periods.

It is of considerable interest that the small amount of longitudinal steel in the sections reinforced with the 45-pound wire fabric was able to cause contraction of the entire 500-foot section without steel failure. By any reasonable assumptions this would indicate that, when such contraction occurred, the coefficient of subgrade resistance was very much lower than the value of 1.5 assumed when the regular sections were designed. It appears also that the coefficient of resistance is smaller for the slow annual changes in length than it is for the more rapid daily changes, this being in agreement with data obtained on the regular sections.

Part 5.—THE OCCURRENCE OF PUMPING

It is generally conceded that three factors are necessary for the development of pumping at joints or cracks in concrete pavements: (1) frequent repetition of heavy axle loads and accompanying large vertical movements of slab edges; (2) fine-grained subgrade soils; and (3) free water under the pavement slab. Since the experimental sections were constructed as a part of a heavily traveled route, one of the factors, repetition of heavy axle loads, is always present. Moreover, the pavement was placed on a fine-grained subgrade soil that would be considered conducive

to pumping. The soil analysis, shown in table 8, was based on samples taken from the finished subgrade at intervals of approximately 500 feet. Referring to the average values of the table, it is observed that the combined amount of silt and clay was 65 percent of the total.

Early in the life of the pavement, pumping began to appear at some of the bridge-type joints which were used to create the wider separations between the longer and consequently more heavily reinforced sections. These joints had no medium for load transfer except the steel cover plate. After 10 years most joints of this type were pumping and it has been noted that this action in some instances has resulted in serious faulting with transverse cracking of the forward and sometimes the approach slab. Of special interest is the fact that, in spite of the faulting, the heavy reinforcement has thus far held closed all cracks in the pavement areas adjacent to these joints. After 7 or 8 years pumping was observed at some of the conventional dowel joints which separate the shorter sections. To date, however, the action at these joints has been so slight that faulting is negligible and fracturing of the slabs has not occurred.

At the end of 10 years the performance survey showed that, with two exceptions, the only evidence of pumping in the entire pavement was in the vicinity of the transverse joints. One of the exceptions was the development of pumping at two of the cracks that formed in the sections containing the 32-pound wire fabric. As stated before, the reinforcing steel ruptured at several cracks in these sections, allowing wide separations. Pumping appeared shortly after the reinforcement failed.

The other exception was the appearance of pumping at one point along the edge of the heavily traveled lane some distance from the end of one of the most heavily reinforced sections. This condition was observed during the survey at the age of 10 years. Mud was not being ejected through any of the cracks, but was appearing at the shoulder and the pavement edge. The cracks in the immediate vicinity were seemingly as tightly closed and as nearly watertight as any in the section. For this reason, it is believed that water reached the subgrade by some other channel, along the pavement edge or possibly through the longitudinal joint. At three consecutive cracks in the immediate vicinity of the pumping, where the crack interval is about 2.5 feet, the edges of the cracks on the pavement surface are raveled or chipped to a more pronounced extent than at the other cracks in the section, indicating that the segments of pavement have been deflected considerably by heavy loads in spite of the presence of heavy longitudinal steel. After a few more years of service this condition may reach a point where some form of maintenance will be necessary.

Pumping Absent at Cracks

The complete absence of pumping at the vast number of transverse cracks in these sections, on a pumping type of soil, is evidence of the effectiveness of the reinforcing steel in holding tightly together the segments of the sections. Closed cracks not only minimize the leakage of free water to the subgrade but, by transferring load, minimize slab deflections as well.

Table 8.—*Subgrade soil data*

	Silt	Clay	Liquid limit	Plasticity index	Moisture content: 0–3 inches below surface	Moisture content: 3–12 inches below surface	Moisture content: 12–24 inches below surface
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Maximum	65	[1] 26	52	26	22.6	21.0	27.5
Minimum	20	7	19	4	6.1	8.9	8.1
Average	48	17	33	12	12.8	15.5	17.1

[1] This maximum percentage was exceeded in two instances; however, these cases were not considered as representative of the entire project.

Periodic observations of the weakened-plane warping joints of the special sections have shown that pumping developed at 10 of the 11 surface-type joints at which the wire-fabric reinforcement failed. In all 10 cases the action of pumping began shortly after steel failures were noted. Conversely, pumping did not appear as long as the wire fabric crossing the warping joints remained structurally sound; or at any of the seven submerged-type joints at which the reinforcing steel failed.

The preceding observations indicate that the entrance of free water to the subgrade and large slab deflections have activated pumping. Relatively wide separations (up to three-eighths inch) developed at all joints at which the reinforcement failed, thus reducing or destroying the effectiveness of aggregate interlock, and impairing the sealing of the surface-type joints. Of particular interest is the complete absence of pumping at the submerged-type joints, even those at which the wire fabric failed. When these joints were installed a copper seal which enveloped the bottom parting strip was incorporated in the design. Apparently these seals are still functioning as planned, despite the wide separation that has developed at the joints where the longitudinal steel has failed.

The effect of repetition of heavy axle loads on the development of pumping is clearly revealed in this investigation. As mentioned previously, the experimental pavement is one-half of a divided highway, with the result that the right-hand lane carries a greater number of heavy vehicles. At the end of 10 years the performance survey disclosed only one case of pumping in the left-hand or passing lane of the pavement, in contrast to the condition in the right-hand lane as just described.

Part 6.—PAVEMENT SMOOTHNESS

The common goal of all pavement design is a continued smooth riding surface, economics being, of course, a limiting factor. To evaluate the riding quality of the experimental sections, an instrument for indicating the relative roughness of road surfaces was used. With this device, which was developed some years ago by the Bureau of Public Roads,[5] it is possible to compare the surface roughness of the various sections by means of a roughness index, expressed in inches per mile of pavement. Low index values, of course, represent smooth pavement.

The basic set of data indicating relative values of surface roughness of these experimental sections was obtained in August of 1940, less than 2 years after construction. At that time roughness indices were determined only for the sections in the heavily traveled or right-hand lane of the pavement, it being presumed that, early in the life of the pavement, the surface of both lanes would be equally smooth. A second set of data was obtained in August of 1949, these data including both the heavily traveled lane and the passing lane so that the effect of traffic on surface roughness could be ascertained. Eliminated from these latter data was the localized condition of roughness found at locations where a pair of bridge-type joints were spaced 10 feet apart.

Average values of the surface roughness of short, intermediate-length, and long sections are given in table 9, the sections being grouped according to the range in lengths designated earlier in the report. All values were obtained with the single wheel of the roughness measuring vehicle traversing approximately a midlane path.

It is well to point out that, as a result of experience gained in using this equipment over many hundreds of miles of pavements of all types, it has been found that pavements with indices of the order of 80 to 120 have surfaces that would be classed as smooth riding.

The data of table 9 show that initially the pavement as a whole was very smooth indeed, indicating that the construction and, particularly, the finishing were unusually good. The fact that little difference was observed in the roughness indices of the three groups of sections suggests that, with proper care

Table 9.—*Roughness indices classified by length of sections*

Range in section length	Roughness index, in units per mile: 1940 survey, right lane	Roughness index, in units per mile: 1949 survey, Right lane	Roughness index, in units per mile: 1949 survey, Left lane
Feet			
0–120	89	131	129
120–470	86	130	124
470–1,310	90	126	124

[5] *Standardizable equipment for evaluating road surface roughness*, by J. A. Buchanan and A. L. Catudal; PUBLIC ROADS, vol. 21, No. 12, Feb. 1941.

During installation and finishing, the spacing of expansion joints need not affect the initial riding quality of concrete pavements.

A comparison of the roughness indices (table 9) determined in 1949 with those of 1940 indicates a marked increase in the surface roughness of all three groups of sections, the percentage increase in the units per mile being 47, 51, and 40, respectively, for the short, intermediate-length, and long sections of the heavily traveled lane. Even with this large percentage increase, the data indicate that the surface of the regular sections after 10 years of service is no rougher than some new pavements as constructed.

Comparison of Lanes

Only a slight tendency is noted, however, for the surface of the heavily traveled lane to become rougher, with time, than that of the passing lane. Also, in both lanes there is a slight but only a slight tendency for the pavement of the group of long sections to be smoother than that of the short sections, which contain relatively few cracks. It is apparent from these observations that, to date, traffic has had little effect on the increase in surface roughness; and that the many cracks that formed in the long sections have not affected the riding quality of the pavement.

In connection with the observed increase in pavement roughness between 1940 and 1949, it will be recalled that figure 1 shows examples of observed changes in pavement elevation, changes that developed principally through heaving and settlement of the subgrade and that undoubtedly account for part, at least, of the increase in surface roughness.

In table 10 are given the roughness indices for sections containing the various percentages of longitudinal steel for each of the three types of reinforcement. These data, although showing no particular trends for the factors involved, do show that a narrow range in the roughness indices of the various sections existed initially and still exists. This implies that all of the sections have remained structurally intact.

Table 10.—*Roughness indices classified by type and percentage of longitudinal steel*

Type of reinforcement and percentage of longitudinal steel in section	Roughness index, in units per mile		
	1940 survey, right lane	1949 survey	
		Right lane	Left lane
Percent			
Rail-steel bars (deformed):			
1.82	99	124	127
1.02	85	123	121
0.45	85	127	124
0.26	85	128	121
0.11	84	130	122
Billet-steel bars (deformed):			
1.82	85	128	123
1.02	78	130	123
0.45	90	130	121
0.26	88	125	128
0.11	91	135	129
Wire fabric (cold-drawn wires):			
0.42	90	137	129
0.28	90	133	124
0.24	89	131	127
0.17	89	137	139
0.11	84	120	124
0.07	98	134	126
Average	88	129	125

Roughness indices of the four 500-foot special sections were obtained at the same time as those of the regular sections. These data, given in table 11, are listed in accordance with the distinguishing features of the special sections.

At the time of the 1940 roughness survey the surfaces of the special sections were as smooth as those of the regular sections, the average index for the two types of pavement being 89 and 88 respectively. This indicates that the surface-type, weakened-plane joints were finished with great care and that the rather wide and meandering cracks which formed above the parting strips of the submerged-type joints did not impair the riding quality of the pavement at that time.

Table 11.—*Roughness indices of the four 500-foot special sections* [1]

Type of joint and weight of reinforcement	Shear bars	Roughness index, in units per mile		
		1940 survey, right lane	1949 survey	
			Right lane	Left lane
Lb. per 100 sq. ft.				
Submerged type:				
91	Yes	100	134	137
91	No	90	134	127
45	Yes	90	141	148
45	No	90	134	148
Surface type:				
91	Yes	79	141	116
91	No	90	120	169
45	Yes	79	148	127
45	No	95	218	137

[1] Each roughness index is based on 250 feet of pavement.

In 1949 the surfaces of the special sections were, in general, somewhat rougher than those of the regular sections. This greater increase in roughness is probably the result of the conditions at some of the joints that were described earlier, conditions that were not present at the time of the 1940 measurements. An example is the large increase in the roughness index of the half of the section with surface-type joints and containing the 45-pound fabric and no shear bars. It was in this half of the section that pumping developed at the joints where the reinforcing steel failed. This action has resulted in some tilting of the 10-foot slab units and a consequent faulting at the joints until, at present, this particular part of the pavement is quite rough.

New Publications

The *Highway Capacity Manual*, by the Committee on Highway Capacity, Department of Traffic and Operations, Highway Research Board, has now been published in book form by the Bureau of Public Roads and is available from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 65 cents.

The report, concerning the effectiveness of various highway facilities in the service of traffic and involving the many elements of highway design, vehicle and driver performance, and traffic control, was originally published in PUBLIC ROADS, vol. 25, Nos. 10 and 11 (October and December 1949) under the title "Highway Capacity: Practical Applications of Research." The 147-page reprint, in convenient 6- by 9-inch book size, will be a valuable addition to the library of every design and traffic engineer.

The manual encompasses both rural and urban facilities. Its seven major sections provide extensive discussion of maximum observed traffic volumes, fundamentals of highway capacity, roadway capacities for uninterrupted flow, signalized intersections, weaving sections and unsignalized cross movements, ramps and their terminals, and the relation of hourly capacities to annual average volumes and peak flows.

The study is based on a great mass of field observations, made available by the cooperative efforts of the Bureau of Public Roads, the Highway Research Board Committee on Highway Capacity, and many State, county, and city engineers, intensively applied in many places and for a number of years. From the vast grist of basic data, painstakingly analyzed, has been evolved this practical guide to rational methods for the determination of highway capacity, essential in the sound economic and functional design of new highways and in the adaptation to present or future needs of the many existing roads and streets which must continue in use for extended periods of time.

***Highway Statistics, 1948*, is also now available from the Superintendent of Documents. The fourth in an annual series, this publication presents statistical information of general interest on the subjects of motor fuel, motor vehicles, highway-user taxation, State highway finance, and highway mileage for the year 1948. The summary bulletin reporting similar information over periods of 20 to 50 years, up to 1945, is a valuable adjunct to the annual series. These publications are sold by the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at the following prices:**

	Cents
Highway Statistics, 1948	65
Highway Statistics, 1947	45
Highway Statistics, 1946	50
Highway Statistics, 1945	35
Highway Statistics, Summary to 1945	40

U. S. GOVERNMENT PRINTING OFFICE: 1950

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Work of the Public Roads Administration:
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Economic and Statistical Analysis of Highway Construction Expenditures. 15 cents.

Electrical Equipment on Movable Bridges (No. 265T). 40 cents.

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Guides to Traffic Safety. 10 cents.

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Highway Bond Calculations. 10 cents.

Highway Bridge Location (No. 1486D). 15 cents.

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Highway Practice in the United States of America. 50 cents.

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Highway Statistics, 1946. 50 cents.

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Highways of History. 25 cents.

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Public Control of Highway Access and Roadside Development. 35 cents.

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Roadside Improvement (No. 191MP). 10 cents.

Specification for Construction of Roads and Bridges in National Forests and National Parks (FP-41). $1.25.

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Tire Wear and Tire Failures on Various Road Surfaces. 10 cents.

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Single copies of the following publications are available to highway engineers and administrators for official use, and may be obtained by those so qualified upon request addressed to the Bureau of Public Roads. They are not sold by the Superintendent of Documents.

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No. 369 . . . Studies of Highway Skew Slab-Bridges with Curbs. Part I: Results of Analyses.

No. 375 . . . Studies of Slab and Beam Highway Bridges. Part II.

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF FEBRUARY 28, 1950

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$15,347	$13,966	$7,142	392.4	$7,143	$3,518	141.7	$10,731	$6,160	225.6	$31,840	$16,820	759.7
Arizona	2,064	6,193	4,324	165.8	1,451	1,011	36.0	5,019	3,425	55.6	12,663	8,760	257.4
Arkansas	6,870	10,224	5,635	308.0	6,662	3,232	158.5	10,863	5,488	222.8	27,749	14,355	689.3
California	7,009	47,449	17,969	287.6	3,937	1,977	33.6	36,207	18,596	168.9	87,593	38,542	490.1
Colorado	7,966	1,378	761	23.7	5,398	2,886	88.8	11,555	6,838	229.3	18,331	10,485	341.8
Connecticut	5,362	8,198	3,733	17.8	2,132	1,416	3.2	5,517	2,986	6.7	15,847	8,135	27.7
Delaware	3,040	2,251	1,123	44.2	2,072	1,031	33.0	2,008	954	23.0	6,331	3,108	100.2
Florida	10,038	11,475	6,225	192.3	4,094	2,270	96.8	8,454	3,983	216.3	24,023	12,478	505.4
Georgia	7,298	18,974	9,943	672.1	4,766	2,386	188.9	41,337	19,541	604.8	65,077	31,870	1,465.8
Idaho	5,885	10,475	6,601	434.3	1,198	686	42.7	4,363	2,675	92.1	16,036	9,962	569.1
Illinois	27,464	46,270	24,068	439.0	11,367	6,163	122.9	42,377	20,314	295.8	100,014	50,545	857.7
Indiana	18,137	20,611	10,235	104.4	7,114	3,426	34.7	9,948	5,781	21.7	37,673	19,442	160.8
Iowa	7,541	15,119	7,085	858.9	3,304	1,596	138.7	14,701	7,481	397.2	33,124	16,162	1,394.8
Kansas	7,324	11,081	5,529	1,091.7	7,536	3,750	486.1	13,283	6,834	857.3	31,900	16,113	2,435.1
Kentucky	3,006	22,907	11,361	417.1	5,020	2,522	105.7	12,170	5,955	179.5	40.097	19,838	702.3
Louisiana	8,691	23,028	11,172	239.3	6,772	3,768	51.6	14,186	6,308	136.4	43,986	21,248	427.3
Maine	4,261	7,285	3,961	102.3	219	115	4.2	5,731	2,850	77.7	13,235	6,926	184.2
Maryland	3,304	8,266	3,970	54.9	409	204	6.1	15,752	7,671	51.7	24,427	11,845	112.7
Massachusetts	1,258	28,300	13,358	41.1	18,557	8,616	20.8	37,069	17,722	22.7	83,926	39,696	84.6
Michigan	11,954	18,223	9,420	511.0	5,748	2,886	102.1	37,581	15,043	165.8	61,552	27,349	778.9
Minnesota	6,617	15,453	8,266	699.2	7,863	4,076	192.1	12,865	6,572	142.9	36,181	18,914	1,034.2
Mississippi	15,100	1,646	822	61.9	250	123	6.4	9,568	5,016	208.9	11.464	5,961	277.2
Missouri	7,992	37,291	19,394	1,001.7	14,409	5,822	250.8	23,331	12,265	480.9	75,031	37,481	1,733.4
Montana	12,436	10,503	5,222	313.2	2,406	1,427	100.9	9,968	6,268	219.4	22,877	12,917	633.5
Nebraska	8,231	18,139	9,400	700.8	5,317	2,967	78.2	8,946	5,171	186.1	32,402	17,538	965.1
Nevada	5,021	3,809	3,152	94.8	691	568	24.5	3,512	2,861	100.4	8,012	6,581	219.7
New Hampshire	2,423	6,938	3,435	66.8	646	304	2.0	1,532	741	10.3	9,116	4,480	79.1
New Jersey	3,760	11,579	5,690	20.4	3,106	1,553	1.4	18,637	8,881	29.8	33,322	16,124	51.6
New Mexico	2,333	11,589	7,402	428.1	3,145	2,028	73.6	7,037	4,583	211.6	21,771	14,013	713.3
New York	53,538	57,711	30,610	241.0	12,831	7,585	25.7	81,688	40,303	152.8	152,230	78,498	419.5
North Carolina	12,527	12,097	5,996	278.3	5,590	2,595	132.9	17,340	8,457	375.3	35.027	17,048	786.5
North Dakota	4,625	11,203	5,772	1,372.3	4,093	2,124	219.2	6,354	3,354	465.5	21,650	11,250	2,057.0
Ohio	9,214	74,131	35,945	321.2	13,612	6,460	94.0	28,326	14,015	116.7	116,069	56,420	531.9
Oklahoma	1,160	35,248	18,339	767.3	6,697	3,202	181.8	10,251	5,304	268.3	52,196	26,845	1,217.4
Oregon	2,276	9,418	5,504	164.7	2,706	1,579	88.8	10,176	5,428	86.7	22,300	12,511	340.2
Pennsylvania	16,821	33,296	16,648	93.8	9,277	4,858	37.7	69,398	33,972	151.8	111,971	55,478	283.3
Rhode Island	1,627	7,271	3,653	37.2	7,000	4,023	1.1	4,031	1,576	4.2	18,302	9,252	42.5
South Carolina	5,390	8,046	4,057	362.6	1,420	775	101.7	11,023	5,544	393.5	20,489	10,376	857.8
South Dakota	2,820	14,690	8,538	1,417.4	2,534	1,713	273.8	5,183	3,173	330.9	22,407	13,424	2,022.1
Tennessee	7,040	12,232	6,146	272.6	9,298	4,444	168.5	14,888	6,858	190.8	36,418	17,448	631.9
Texas	15,852	7,539	3,945	639.4	13,580	6,640	652.5	62,154	28,863	1,516.5	83,273	39.448	2,808.4
Utah	5,281	2,343	1,732	88.0	1,502	1,121	40.0	5,320	3,863	125.9	9,165	6,716	253.9
Vermont	1,165	4,387	2,377	44.3	115	56	.1	4,271	1,963	40.6	8,773	4,396	85.0
Virginia	14,089	14,429	7,248	394.6	4,597	2,235	182.0	12,620	6,257	178.6	31,646	15,740	755.2
Washington	3,520	15,657	6,902	199.7	6,550	2,724	74.4	13,263	6,117	64.9	35.470	15,743	339.0
West Virginia	2,877	17,212	7,216	196.7	4,067	2,139	38.9	6,898	3,403	33.7	28,177	12,758	269.3
Wisconsin	13,006	24,137	12,421	479.5	1,566	741	46.7	12,112	6,029	286.5	37,815	19,191	812.7
Wyoming	1,310	6,065	3,778	220.5	1,636	1,120	25.4	5,782	3,746	210.5	13,483	8,644	456.4
Hawaii	2,042	6,919	3,130	32.7	4,049	1,871	8.9	4,289	2,208	15.4	15,257	7,209	57.0
District of Columbia	2,823	3,370	1,897	.5				11,091	5,510	1.9	14,461	7,407	2.4
Puerto Rico	5,420	8,868	4,010	33.9	1,479	635	7.3	9,797	3,610	48.0	20,144	8,255	89.2
TOTAL	412,155	834,889	422,262	17,443.0	256,931	130,967	5,027.4	840,503	418,516	10,700.2	1,932,323	971,745	33,170.6

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